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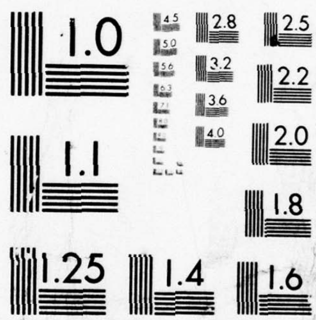

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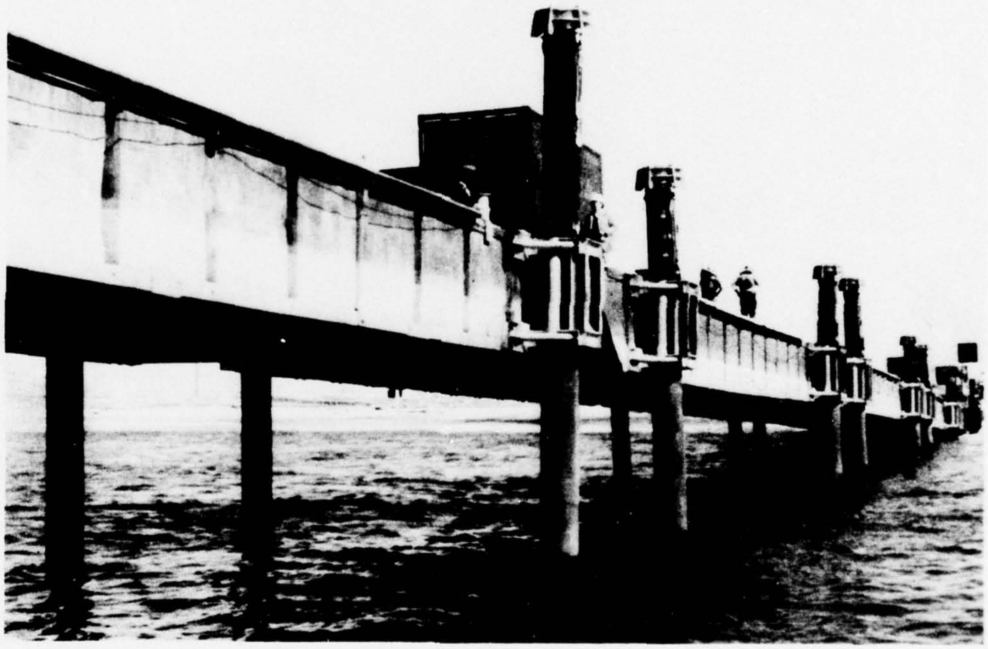

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CONTAINER OFF-LOADING AND TRANSFER SYSTEM (COTS)

Advanced Development Tests of Elevated Causeway System

VOLUME III - ELEVATED CAUSEWAY STRUCTURE

(10) by B. R. Karrh

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Analyses and tests of the elevated structure to ascertain the operating capabilities and limitations are described. A structural analysis technique was formulated for the unique modular causeway structure. To compute stresses due to single and combination loads, graphical solutions are presented. A failure analysis of the structure was made to determine operational safety factors. Structural aspects of an elevated crane platform and the impact of the 40-foot (12.2-m) container on the elevated causeway system are discussed.

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SECTION 1

INTRODUCTION

1.1 SCOPE

The advanced development tests for the elevated causeway were performed to evaluate system hardware, using an adequate number of pontoon sections, existing military lighters and trucks, and 8 x 8 x 20-foot (2.4 x 2.4 x 6-m) commercial containers. The equipment tested included four specially assembled NL pontoon pierhead sections with internal spudwells, five existing pontoon sections equipped with external spudwells, two types of plastic foam fender systems, three types of Navy lighters, one type of Marine Corps tractor/trailer, a turntable, and two types of commercial container handlers. In addition, other selected hardware items were evaluated during the operation. Timing data were taken at pertinent points of the operation; however, this information was considered to be secondary to determining any operational limitations, proper procedures, and problems requiring further development efforts.

1.2 BACKGROUND

DOD planning for the logistics support to sustain major contingency operations, including amphibious assault operations and Logistics-Over-the-Shore (LOTS) evolutions, relies extensively on the utilization of U.S. Flag commercial shipping. Since the mid-1960s commercial shipping has been steadily shifting towards containerships, Roll-On/Roll-Off/ (RO/RO) ships, and bargeships (e.g., LASH, SEABEE). By 1985 as much as 85% of U.S. Flag sealift capacity may be in container-capable ships – mainly non-self-sustaining (NSS) containerships. Such ships cannot operate without extensive port facilities.

Amphibious assault and/or LOTS operations are usually conducted over undeveloped beaches, and expeditious response times preclude conventional port development. The handling of containers in this environment presents a serious problem. This problem is addressed in the overall DOD Over-the-Shore Discharge of Cargo (OSDOC) efforts, which involve developments by the Army, Navy, and Marine Corps. Guiding policy is documented in the "DOD Project Master Plan for Surface Container Supported Distribution System" and the OASD I&L system definition paper "Over-the-Shore Discharge of Cargo (OSDOC) System."

In response to the DOD Master Plan, Navy Operational Requirement (OR-YSL03) has been prepared for an integrated Container Off-Loading and Transfer System (COTS) for discharging container-capable ships in the absence of port facilities. The COTS Navy Development Concept (NDCP) No. YSL03 was promulgated July 1975, and the Navy Material Command was tasked with development. The Naval Facilities Engineering Command has been assigned Principal Development Activity (PDA) with the Naval Sea Systems Command assisting.*

The COTS advanced development program includes the ship unloading subsystem, the ship-to-shore subsystem, and common system elements. The ship unloading subsystem includes: (a) the development of Temporary Container Discharge Facilities (TCDF) employing merchant ships and/or barges with add-on cranes and support equipment to off-load non-self-sustaining containerships alongside; (b) the development of Crane on Deck (COD) techniques and equipment for direct placement of cranes on the decks of NSS containerships to render them self-sustaining in an expedient manner; (c) the development of equipment and techniques to off-load RO/RO ships offshore; and (d) the development of interface equipment and techniques to enable ship discharge by helicopters (either existing or projected in other development programs).

The ship-to-shore subsystem includes the development of elevated causeways to allow cargo handling over the surfline and development of self-propelled causeways to transport cargo from ships to the shoreside interface.

The commonality subsystem includes: (a) the development of wave-attenuating Tethered Float

*NAVFAC Program Plan for Container Off-Loading and Transfer System (COTS) of April 1977.

Breakwaters (TFB) to provide protection to COTS operating elements; (b) the development of special cranes and/or crane systems to compensate for container motion experienced during afloat handling; (c) the development of transportability interface items to enable transport of essential outsized COTS equipment on merchant ships – particularly bargeships; and (d) the development of system integration components, such as moorings, fendering, communications, and services.

The Civil Engineering Laboratory (CEL), Port Hueneme, California, was designated by the Naval Facilities Engineering Command (NAVFAC) as the responsible laboratory for the ship-to-shore subsystem. The five-volume report covers only that portion of the ship-to-shore subsystem related to the elevated causeway components and associated container-handling operations.

CEL planned the elevated causeway tests in two phases. The first phase tests conducted by CEL at Point Mugu, California, were designed to investigate operational and structural capabilities of the NL elevated causeway and to develop operational procedures. No container-handling tests were included in this phase. Tests were conducted from 16 June to 16 July 1975.

The Phase II tests were designed to be conducted by military operators, i.e., PHIBCB-ONE and ACU-ONE, Coronado, California, to determine operational limitations and any further development requirements. Container-handling operations were included in these tests. The pier was elevated by PHIBCB-ONE on Silver Strand Beach, Green Beach Two, at coordinates 32°39'08" latitude, 117°09'25" longitude, beginning 12 November 1975 and finishing on 26 November 1975. A survey of the landing site showed a beach gradient of about 1:30 and a water depth of 20 feet (6 m) (at zero tide) 600 feet (183 m) offshore. Container-handling operations began on 2 December and were completed on 5 December 1975. The container-handling crane was positioned on the pierhead on 1 December. The pier was left elevated until 5 January 1976 to check for piling settlement and to provide an opportunity for the pier to encounter rough seas, and then was disassembled from 5 January to 10 January 1976. A 16-mm, color, sound movie has been prepared covering the Phase II tests.

1.3 REPORT COVERAGE

The final documentation of this program, which consists of five volumes, covers results of both the Phase I and Phase II tests.

The approach taken in the development of an elevated causeway for the COTS ship-to-shore subsystem was to expand the capabilities of the existing Navy Lightered (NL) floating pontoon causeway system. This report, Volume III, documents the evolution of the NL pontoon system into a pile-supported structure capable of sustaining equipment and loads associated with container-handling operations. The causeway assembly and the development of ancillary hardware components are reported, structural analyses and field tests are described, failure modes and criteria are examined to determine safety factors for various loads and modes of operation, and applications of the elevated causeway as a crane platform are discussed. Clearly, the system developed by CEL is not the only system that could satisfy the requirement for an over-the-beach capability. Some alternative systems, system components, and comparative costs are presented in Appendix D.

The other four volumes cover the following.

1.3.1 Volume I

This volume summarizes the Phase I and Phase II test operations. Limited test results and a brief description of equipment are presented.

1.3.2 Volume II

This volume reports on the elevating mechanism or lift system used in the tests, and it also presents alternative lift procedures and associated equipment, e.g., description of elevated causeway, pier installation and retrieval (including pro and con of elevating from shore out or offshore in), pile hammer and driving, beach gradients and surveys, ladders and scaffolding, and discussion of multisection lift. A human engineering study was made of both the elevated causeway system hardware and the associated operational procedures. This study was conducted by the Human Factors Technical Division, Naval Electronics Laboratory Center, San Diego, California, and is presented in this volume.

1.3.3 Volume IV

This volume contains a description of the fender system, installation procedures, and lighterage impact tests. Lighterage motions recorded during the container-handling operation are compiled along with environmental data observed during the test periods.

1.3.4 Volume V

This volume details the container handling, i.e., container-transfer rates, container crane, containers, lighters, Marine Corps truck/trailers, pontoon deck reinforcement, turntable, beach ramp, matting, and air-bearing transporter. An alternate method of ship-to-shore container transfer, the load-on/roll-off (Lo/Ro) causeway ferry system using a commercial top-lift loader, was tested and is described in this volume.

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SECTION 2

ELEVATED CAUSEWAY SYSTEM

The system design is based on providing an interface for lighterage at the shore, with an elevated access from the beach over the surf to an elevated pierhead that supports cargo-unloading functions. The major developments include an elevating capability for the existing NL pontoon causeway, a crane installation at the pierhead for off-loading lighterage moored alongside the causeway, fendering to interface between the elevated pierhead and the lighters, turnaround components to handle truck/trailers on the causeway, and a two-way traffic access from the pierhead to the beach.

The system is based on the 3x15 pontoon section, [21 x 90 feet (6.4 x 27.4 m)], which is fundamental to the Amphibious Construction Battalions. To convert the floating 3x15 structure to the elevating mode requires the addition of spudwells, external or internal. The internal spudwells are used in the pierhead where sections are side by side; the external for connecting the causeway to the beach.

The modularity of the elevated system allows field commands to size out the pierhead and approach. As a minimum, four special pierhead sections in a 2-by-2 arrangement should be used to establish the offshore container-handling facility. The 2x2 section pierhead may be expanded in both length and width dimensions to meet requirements. The length of the approach causeways should be selected so as to bridge the surf and locate the pierhead in water sufficiently deep for anticipated lighterage.

2.1 ELEVATED NL PONTOON CAUSEWAY STRUCTURE

Navy lightered (NL) pontoon barges are normally employed as floating causeway structures to unload roll-on/roll-off cargo from LST amphibious transport ships. The elevated causeway application grew out of a military requirement to unload containerized cargo over unimproved beachheads. The elevated structure differs from the floating structure mainly by the addition of internal and external spudwells. Figure 1 depicts the contribution of various elements to the development and use of the elevated structure. The structural analyses deal with the transfer of loads through the elevated structure and into the supporting piles.

Causeway barges are a collection of multiple flotation modules called pontoons. The internally stiffened pontoon, which measures 5 x 5 x 7 feet (1.52 x 1.52 x 2.13 m), is shown in Figure 2. The frame steel box modules are integrated into a barge structure by a system of structural steel angles. Internal spudwells must be incorporated into the causeway at the time of assembly, while external spudwells can be attached to the outside of the causeway structure at any time. The internal spudwells, which substitute one-for-one the P1 pontoons, are completely contained within the causeway section so that causeways can be positioned side by side to form a pierhead wider than one causeway section. External spudwells, which are bolted to the outer causeway angles, provide a clear roadway the full width of the section. The fender system for the elevated pier utilizes both types of spudwells in the installation.

Figure 3 depicts the techniques by which pontoons are bolted into structural angles to form long strings, one pontoon wide, and Figure 4 shows typical bolted connections. Note the 9-inch (0.23-m) opening between pontoons in the string. This opening was provided to make assembly into strings easier, but it acquires a significant structural role, which will be discussed later. Strings are then positioned side by side and bolted together to make up the barge structure.

2.1.1 Phase I Causeway

For the Phase I tests four 3x15 NL pontoon sections and one 1x15 NL pontoon string for fendering were assembled. Existing NL pontoon hardware was integrated with the two types of spudwells to provide the basic elevated causeway structure pictured in Figure 5. Assembly of the first two of the four pierhead sections was accomplished by the

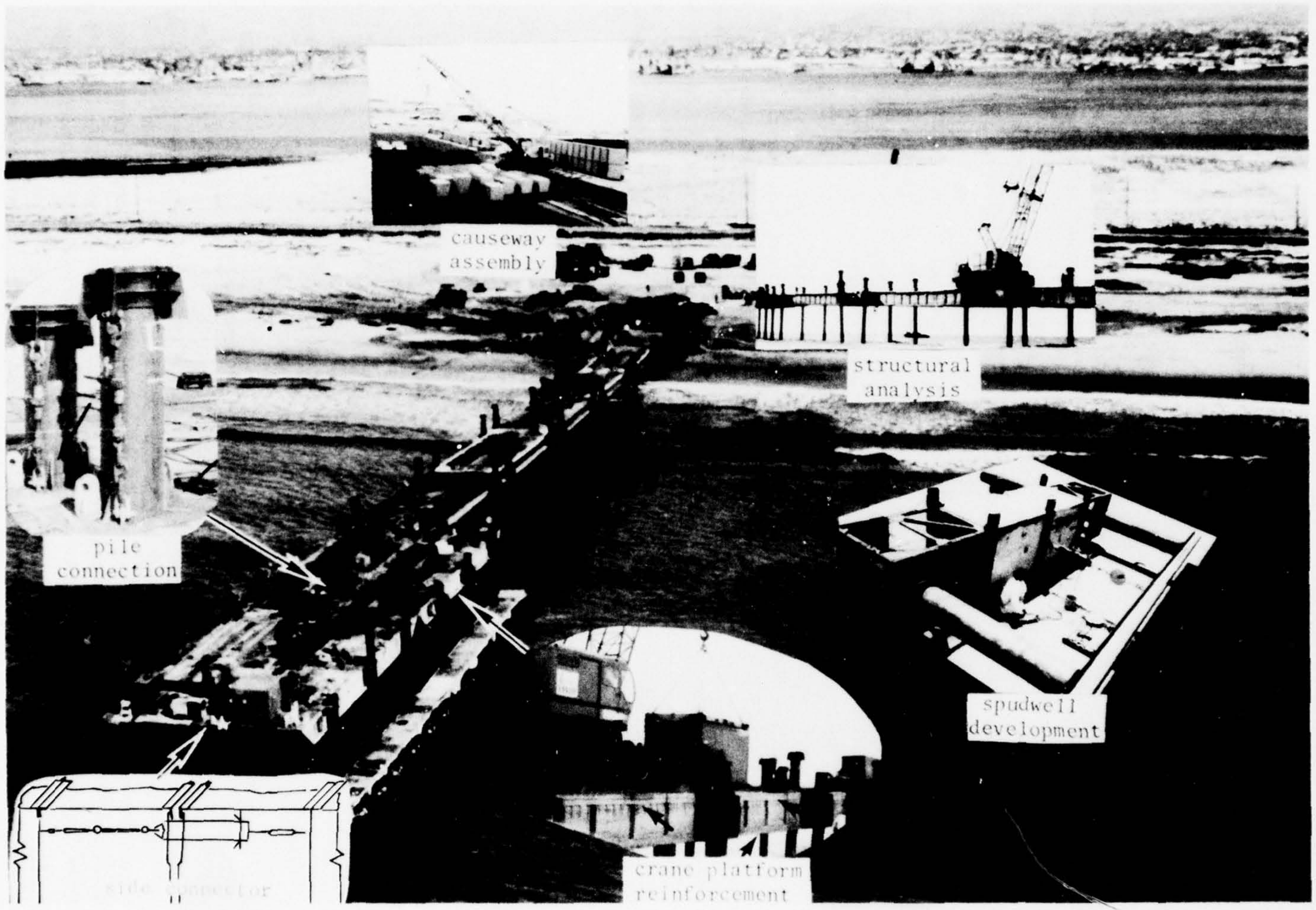


Figure 1. Elevated causeway structure erected at Coronado, California. Analytical and hardware contribution toward the system development are indicated.

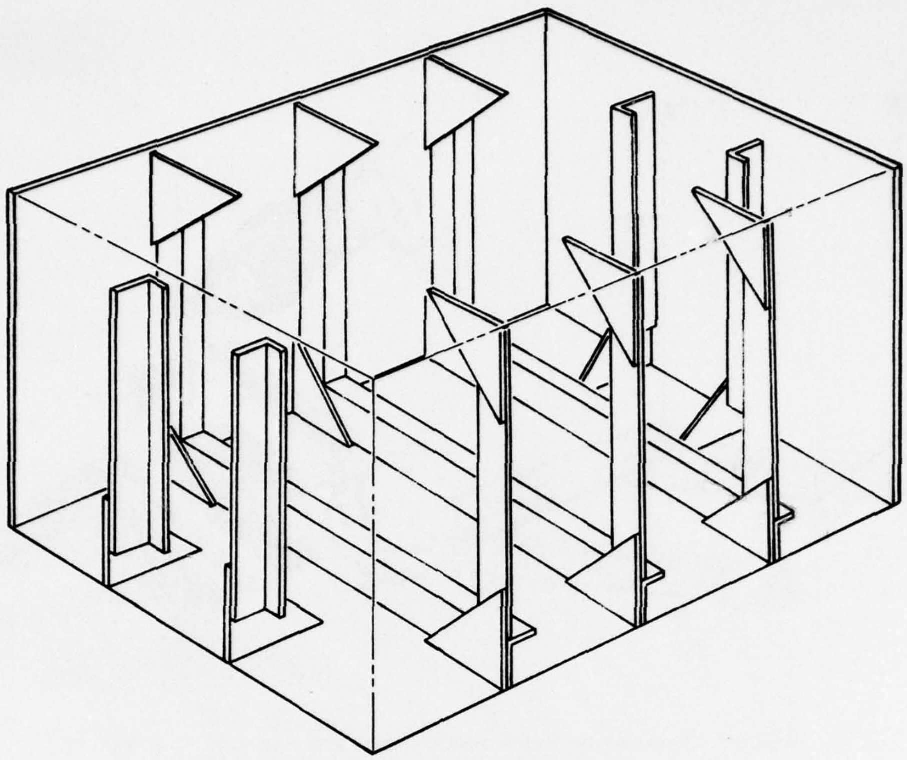


Figure 2. Structural configuration of pontoon modules.

Construction Equipment Department, NCBC, Port Hueneme. The pierhead sections and location of spudwells and end connectors are shown in Figure 6. The first two sections were assembled concurrently using two experienced welders and an inexperienced 16-man crew. Figure 7 shows the causeway assembly area. Additional work required for these elevated sections included welding 24 AP7 plates to angles of each section; reinforcing the outside angles for external spudwells on Sections No. 1, 3, and 4; installing shock absorber assemblies; and applying reinforcement plates between the last two pontoons at each end of all sections. All AP7 plates were welded to the pontoons, which required each pontoon string to be made up separately and then bolted together to make a section. Twenty-one days and 3,100 man-hours were expended on these two sections, considerably more than the 800 man-hours usually required to assemble two standard 3x15 sections. Even though the string assembly method takes longer, the strength of the causeway section is increased.

A third section was assembled by the Construction Equipment Department in 700 man-hours with a nine-man crew over a period of 11 days. The 1x15 fender string was assembled in only 100 hours, indicating that the crew had gained proficiency. The last section was assembled by Public Works Department, NCBC, Port Hueneme, in 14 days with a six-man crew. Irregular working hours prevented an accurate man-hour count. Assembly times and sequences for all sections are shown in Figure 8.

Sections No. 1 and 4 were fitted with both internal and external spudwells. The four external spudwells on Section No. 1 were provided for testing during Phase I, with the six internals to be used to support the container-handling crane during Phase II.* Additionally, three external spudwells were installed on Section No. 4 for the fender system. Internal spudwells adjacent to the fender system were

*Normally, each section of the approach roadway is supported on four piles, while each pierhead section is on six.

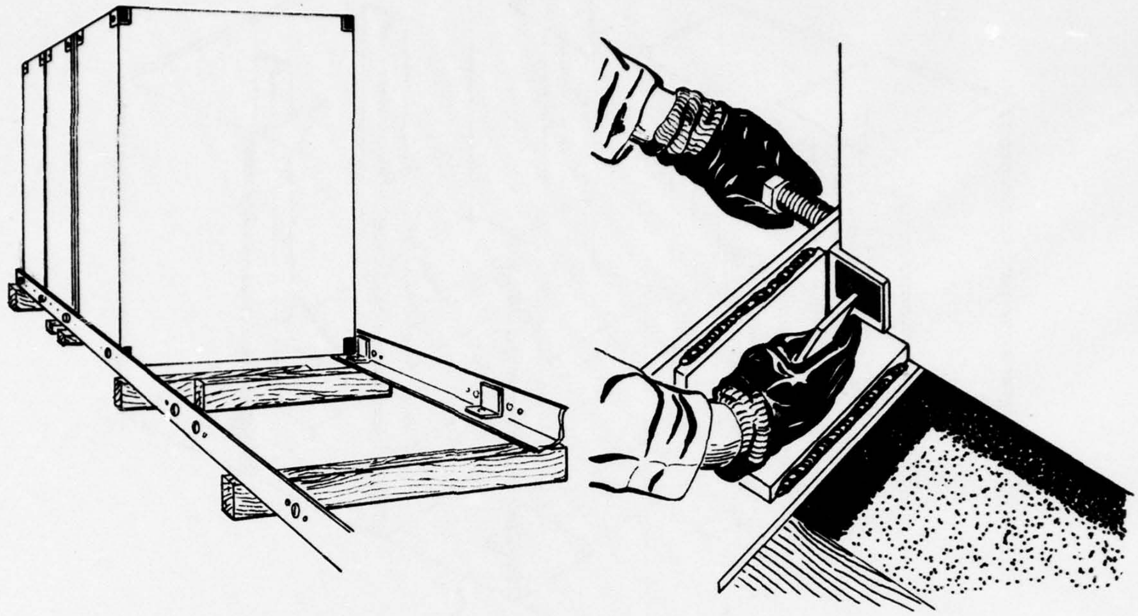


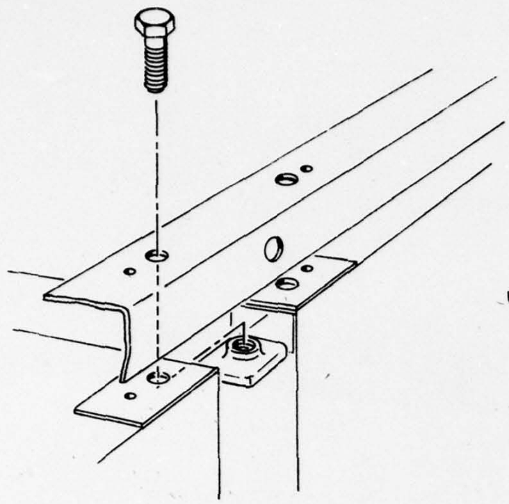
Figure 3. Pontoons are bolted into structural angles to make up strings.

rotated 180 degrees from the normal position. These spudwells were reversed to avoid the possibility of interaction between the support piles and the fender system.

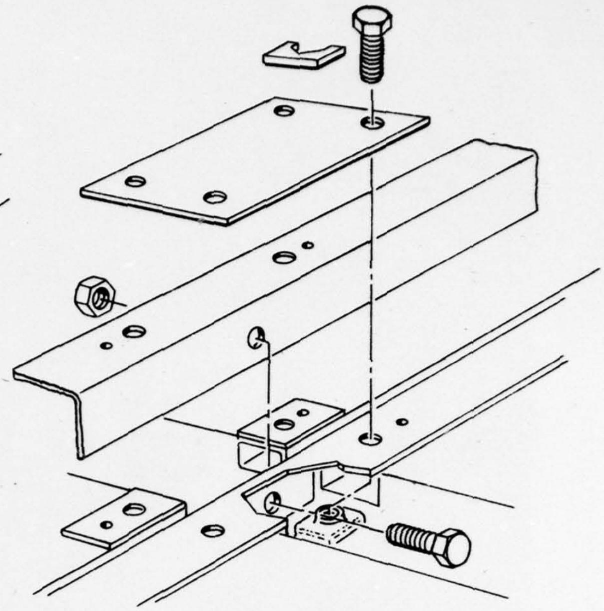
After assembly was completed, a YD floating crane lifted the sections into the water, as shown in Figure 9. Section No. 4 listed slightly due to the external spudwells on one side, with the draft being 19 and 23 inches (0.48 and 0.58 m) on opposite sides; it weighed about 140,000 pounds (63,500 kg). Section No. 1 drafted 21 inches (0.53 m) and weighed about 139,000 pounds (63,000 kg). Caution should be exercised when lifting causeways by chains slung under the angles (shown in Figure 9) because chains frequently deform the vertical angle legs, thereby reducing the angle strength.

For evaluation purposes, two new types of end-to-end connectors, an improved NL pontoon type and a new Flexor type, were installed in the four pierhead sections. Padeye clearances on the NL connector were increased by 1/2 inch (0.127 m) to eliminate the tendency of the link to pop out of the padeye when subjected to compressive loads. A cover plate was added to stiffen the male pipe. Figure 10 shows the male and female NL connectors.

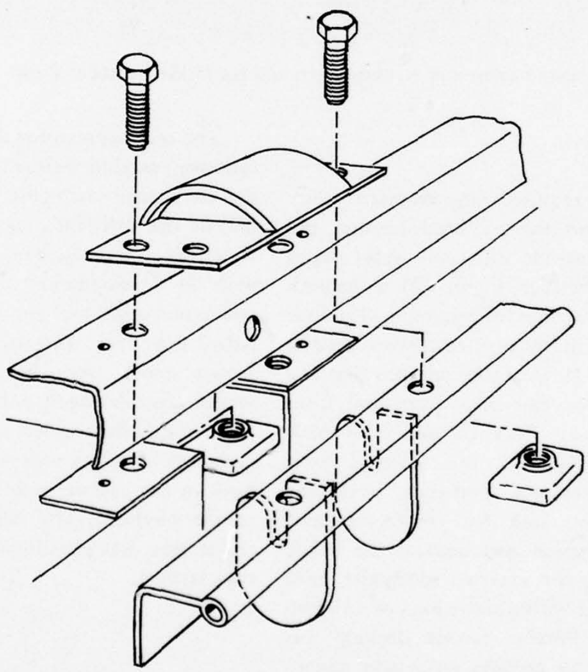
The developmental Flexor end connector, shown in Figure 11, is intended to reduce operational problems with the existing end connectors. Its features are increased safety for personnel and capability to end connect causeways in higher sea states. The Flexor acts like a probe, which can be winched into a faired receiver slot. The Flexor is locked in place by a guillotine-type device. Functionally, the Flexor carries tensile and compressive loads, while a ball and socket arrangement similar to the NL connector absorbs vertical and horizontal shear components. Since the two types of connectors are not compatible, they were located so that two of each connection could be made up using the four sections.



(a) Typical exterior connection between pontoons of a string.



(b) Typical interior connection between adjacent pontoon strings.



(c) Typical connection with launch angle for side-loading.

Figure 4. Schematics of typical connections to bolt pontoons and angles into barge-type structures.

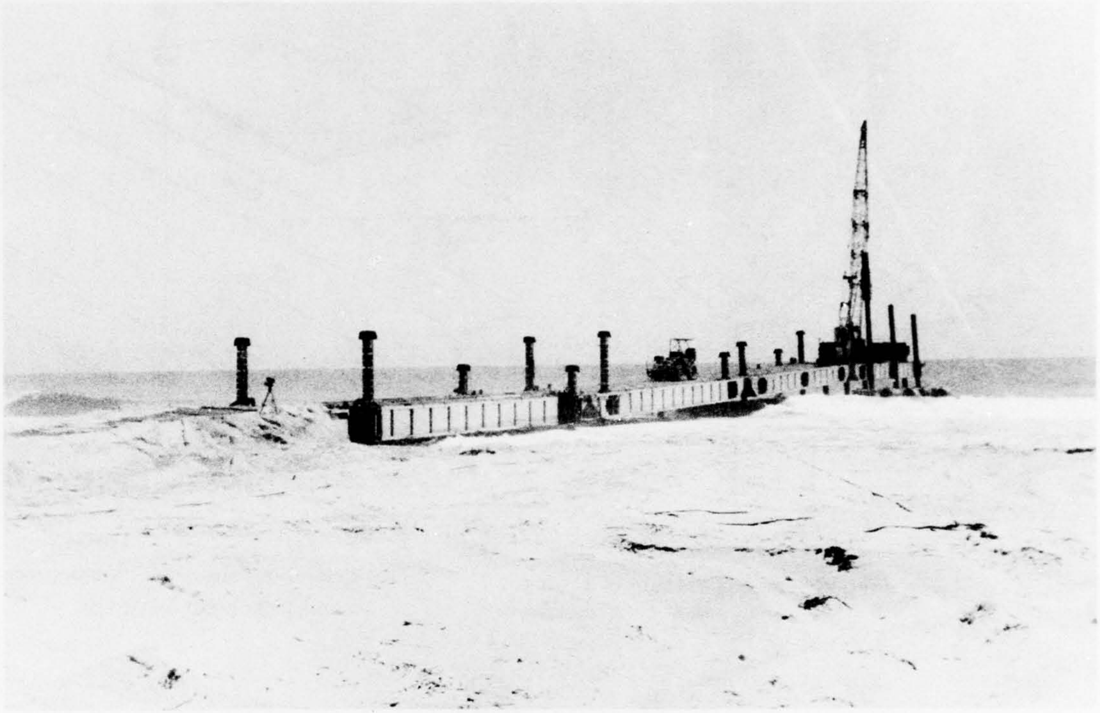


Figure 5. Elevated causeway structure erected for Phase I tests at Point Mugu, California.

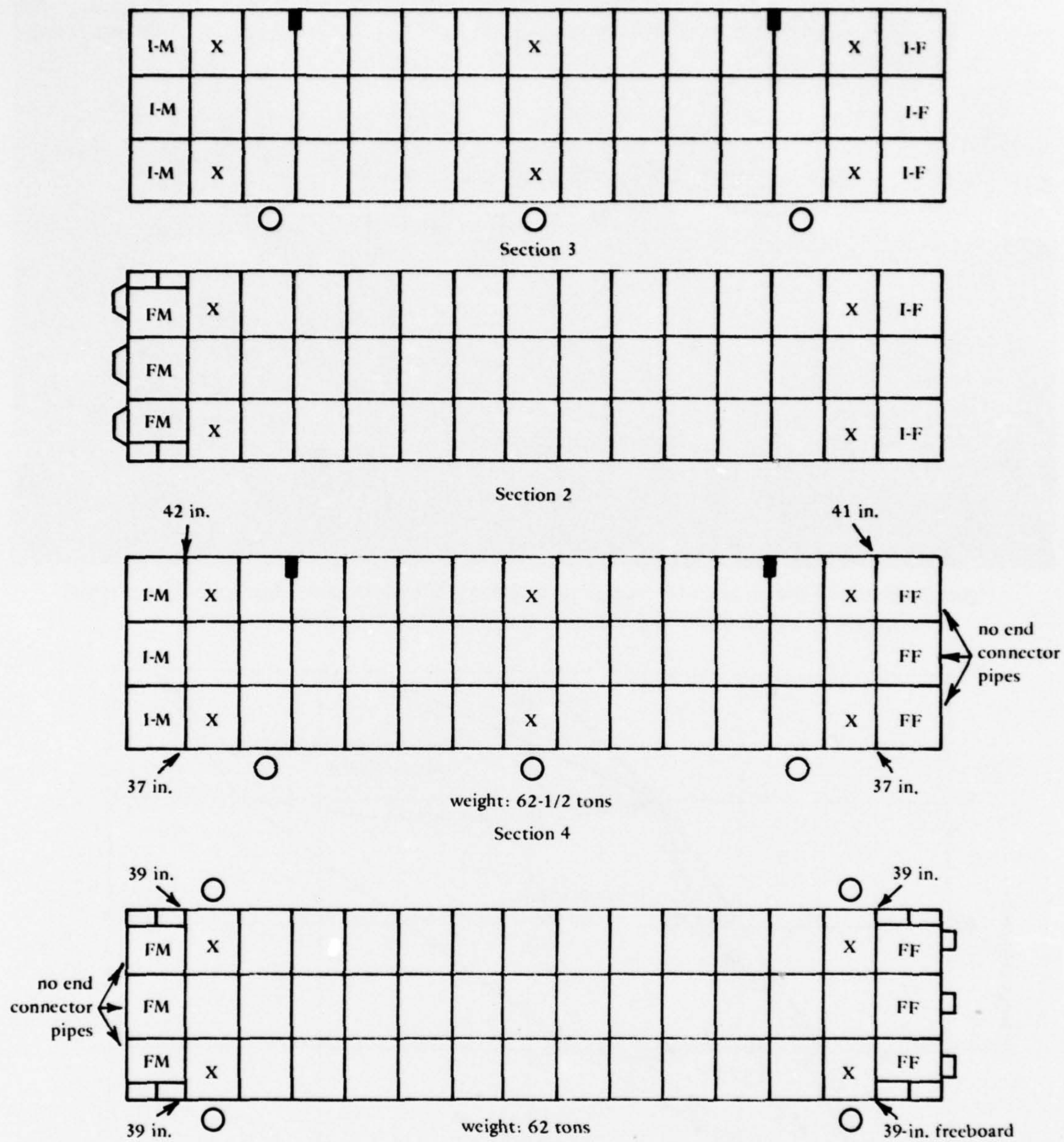
2.1.2 Phase II Causeway

The Phase II tests required nine elevated causeway sections to extend the pierhead beyond the breaking surf and to provide sufficient water depth for lighters alongside the pier (Figure 12), along with two 1x15 NL pontoon strings for fendering. The four sections assembled for the Phase I tests were used as the pierhead for Phase II. Six more sections that had been in service 2 to 4 years were borrowed from PHIBCB-ONE's causeway fleet to be fitted with external spudwells. Five of the sections were modified to receive external spudwells, while the sixth was added too late for reinforcement;* therefore, the sixth section was used at the beach interface. After fitting the external spudwells, each section was prooftested with a static load of 150,000 pounds (68,000 kg). Finally, timber decking was fitted on each section to provide a roadway and to reinforce the pontoon deck.

The original plan for the Phase II tests called for a causeway section seaward of the pierhead to support the air bearing turntable. The turntable section was one of the PHIBCB's standard sections. The section to which it was to mate on the pierhead was fitted with the developmental Flexor end connector. These end connectors are not compatible, but they were lashed with chain and wire rope so that the male and female pipes were held together. The turntable section was damaged during a period of high waves when the lashing failed to hold the ball and socket compression pipes together. The section was towed back to the harbor, and the turntable was transferred to the pierhead. The failure of the improvised end connection was attributed to stretching of the wire rope lashing.

***Reinforcement details are discussed under Spudwell Development.**

Note: 1 in. = 2.5 cm
1 lb = 0.45 kg



Legend

I-F	Improved end connector, female	X	Internal spudwell, reversed pile hole located close to internal angles, weight 3,200 lb
I-M	Improved end connector, male		
FF	Flexor end connector, female		
FM	Flexor end connector, male	O	External spudwell, pontoon angle requires reinforcement, weight 1,200 lb
▮	Side connector		

Figure 6. Location of spudwells and end connectors on pierhead causeway sections.

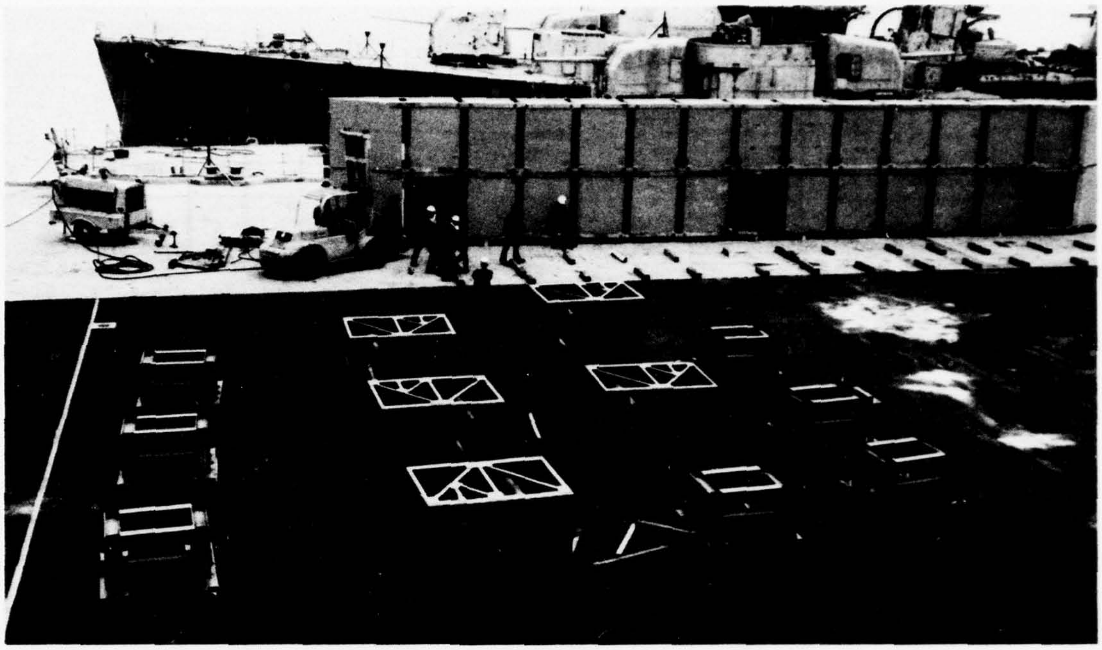


Figure 7. Section 4 during assembly. Large rectangular spudwells in center foreground are internal spudwells and smaller truncated units at right and left are external spudwells.

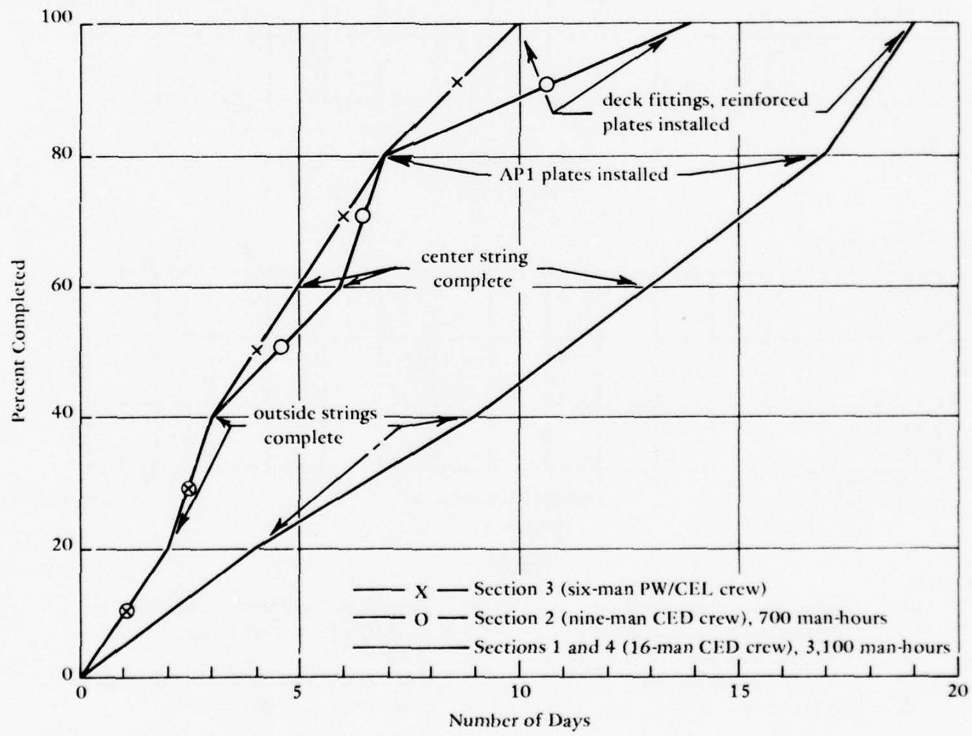


Figure 8. Progressive assembly of four elevated causeway sections.

Figure 9. Section 4 being set into water. Note external spudwells on far side for fender system.

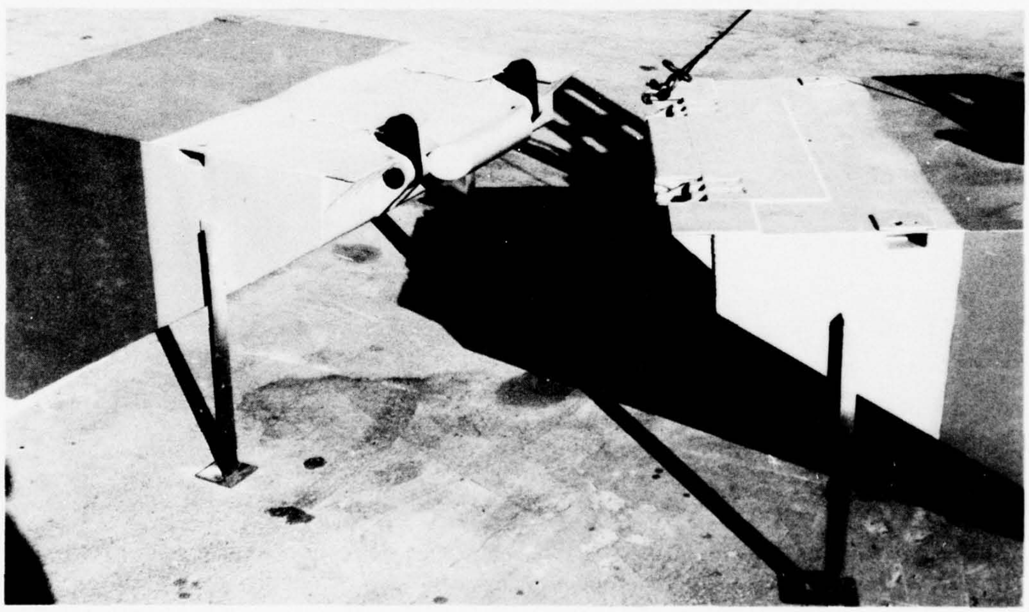


Figure 10. Improved NL pontoon end connectors. The connector was strengthened and padeye/link tolerances corrected to prevent links from popping out during operations.

Figure 11. Developmental Flexor connector being connected in the harbor.

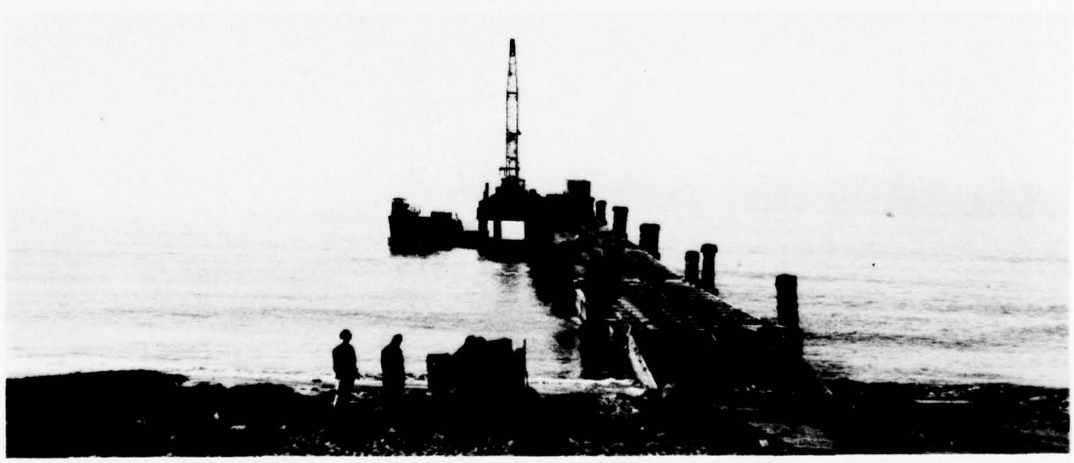


Figure 12. Elevated pier with LCU moored alongside to unload containers. Pier length was adequate for pierhead to extend beyond breaking surf.

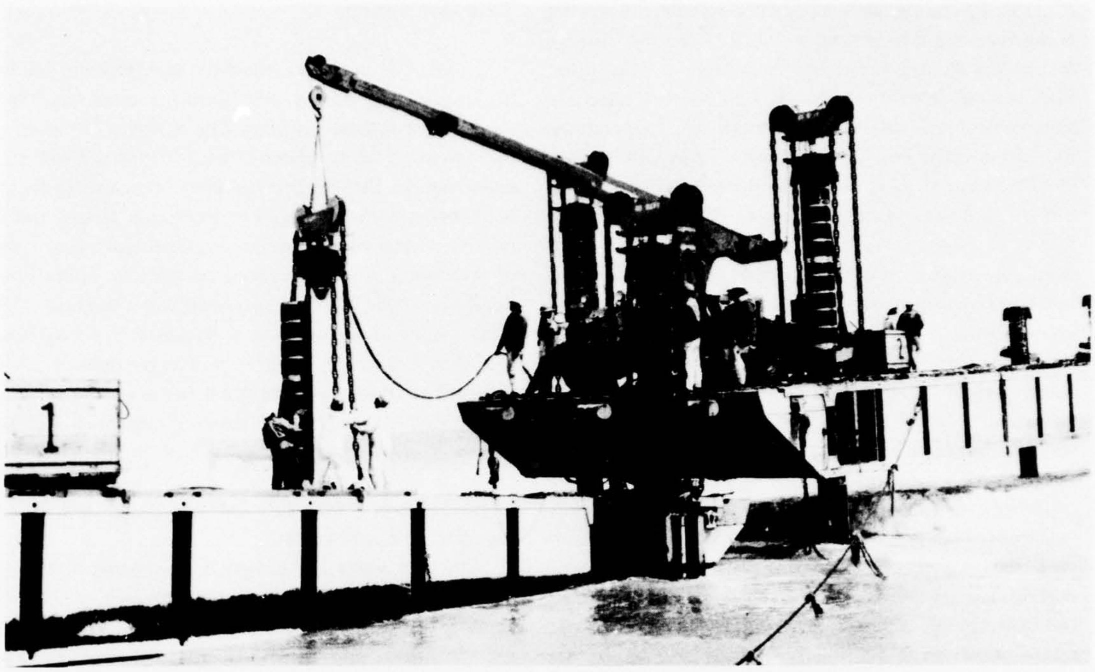


Figure 13. Internal and external spudwells during causeway erection.

2.2 SPUDWELL DEVELOPMENT

When a firm operational requirement for an elevated causeway at an open-beach site was established following OSDOC II, NL pontoon causeways were proposed for the task. NL pontoon causeways were designed as floating structures to discharge vehicular traffic from LST's to the beach. Limited use has been made of the causeways as bridge sections. The modular construction of the NL causeway system provides two spudwell options for supporting the causeway on piles. Internal spudwells replace a standard 5 x 5 x 7-foot (1.52 x 1.52 x 2.13-m) pontoon flotation module, and external spudwells are bolted to the outside angles of the causeway. Both types of modular spudwells are shown in Figure 13. The spudwells interface with the supporting piles.

2.2.1 Design Criteria

Piles and spudwells for the elevated causeway were designed on the basis of gravity and wave loads. Operational requirements call for a maximum water depth of 20 feet (6.1 m) at the pierhead. In this water depth, breaking waves range from 13 to 17 feet (4.0 to 5.2 m) in height, depending on the wave period;* therefore, the most critical pile loading is a 17-foot (5.2-m) breaker in 20 feet (6.1 m) of water. A 20-inch (0.51-m) diameter pipe pile with one-half-inch (12.7-mm) wall thickness satisfies this load requirement. The pile load capacity can be increased up to 50 percent by providing a moment-resistant pile-to-causeway connection instead of a simple, or hinged, support.

The spudwell design criteria were derived from the maximum gravity loads anticipated, plus the ability to resist the moment developed by the design wave environment. The moment capacity of the 20-inch (0.51-m) pile at yield is 437,000 ft-lb (592.60 kN·m). The maximum gravity load expected on the causeway is a combination of the causeway dead load, plus the live load of the largest crane to be driven over the elevated causeway.

Critical gravity loads are possible under two conditions – one with a crane in transit, and one with

*Wiegel, Robert L., Oceanographic Engineering, 1964, Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

a crane operating as a container handler. The first condition would normally occur on the bridge sections with the causeway supported on four piles. The second condition would occur on a specially supported crane platform or pierhead. Calculations indicate a crane transit load limit of 235,000 pounds (107,000 kg) for a four-pile supported causeway with 8-inch (0.20-m) structural angles. For six piles and 8-inch (0.20-m) structural angles, a crane transit load limit of about 340,000 pounds (154,000 kg) is indicated. From a practical standpoint, the heaviest crane within the mission of the elevated causeway is the P&H 6250 truck crane, which can be reduced to a travel weight of 263,000 pounds (119,300 kg) by removing 90,000 pounds (40,800 kg) of counterweight. Even at its travel weight, the 6250 crane exceeds the structural capacity of the four-pile supported causeway; therefore, a six-pile support is required. The combination of causeway dead load plus the maximum expected crane transit load is determined to be no greater than 150,000 pounds (68,000 kg) on a single spudwell/pile, either for the 6250 crane on a six-pile supported section or for lighter cranes on a four-pile supported section.*

The maximum expected outrigger load associated with container-handling operations on a six-pile configuration with a crane weighing 190,000 pounds (86,200 kg) is about 150,000 pounds (68,100 kg). This assumes a container load of 50,000 pounds (22,700 kg) at a radius of 50 feet (15.2 m). To avoid overloading any one spudwell, the design approach is to double up on spudwells so that the load is divided between two or more spudwells, or to position outriggers directly on piles. This was accomplished during the Phase II tests by using the fender piles as secondary gravity load supports.

The analysis summarized above indicates that a design load of 150,000 pounds (68,000 kg) plus 350,000 lb-ft (47.46 kN·m) of moment capacity are appropriate design criteria for the elevated causeway spudwells, because maximum environmental conditions will not occur simultaneous with maximum operational conditions.

A simplified frame analysis of the causeway/pile system subjected to gravity loads indicated that very little moment is transmitted from the causeway to the pile due to the relative flexibility of the piles. However, moment will be transferred to the causeway from the piles, if the piles are loaded by waves.

2.2.2 Description

Spudwells provide one of the components needed to transform floating NL pontoon causeways into elevated structures on piles. The primary function of the spudwell is to transfer loads from the elevated causeway to the supporting piles. The spudwells are of truss-type construction to minimize weight and to provide access to the pile for a connection. Two types of spudwells were developed to provide operational flexibility. The internal spudwell was designed to be used interchangeably with a standard 5 x 5 x 7-foot (1.52 x 1.52 x 2.13-m) pontoon module. The external spudwell was designed to be bolted directly to the outside edges of a floating causeway section. Figure 7 shows the internal and external spudwells prior to installation. Design details are available in CEL drawings 74-25-1F and 74-27-1F, which are provided in Appendix B.

The spudwells are designed to receive a 20-inch (0.51-m) outside diameter pile through a 23-inch (0.58-m) diameter opening top and bottom to accommodate pile rotations up to 3 degrees. The spudwell clearances reduce binding and denting of the piles as the causeways pitch in surf while still in the floating mode.

Installation of internal spudwells must be done at the time the causeway section is assembled; no special preparations or procedures are required. A spudwell is substituted at the desired location for a standard P1 pontoon. Each spudwell reduces the causeway buoyancy about 2 percent and increases the section weight about 1,200 pounds (544 kg). The internal spudwell weighs 3,200 pounds (1,450 kg).

One advantage of external spudwells is the ability to install them on a previously assembled causeway. There are some structural reinforcements required to provide boltholes for the eight standard A6 bolts [1-1/2-inch (38.1-mm), high-strength] used to attach each spudwell to the pontoon angles. Figure 14 shows the basic reinforcements. No buoyancy is lost, but each external spudwell adds about 1,300 pounds (590 kg) of weight.

Spudwells for the elevated causeway were designed to perform the interface functions with the causeway jack-up system. In addition to causeway-to-pile load transfer, the spudwells furnish a lifting mechanism for the hydraulic chain jacks and a padeye system for temporary support of the causeway after elevation.

*Appendix C discusses crane transit loads in greater detail.

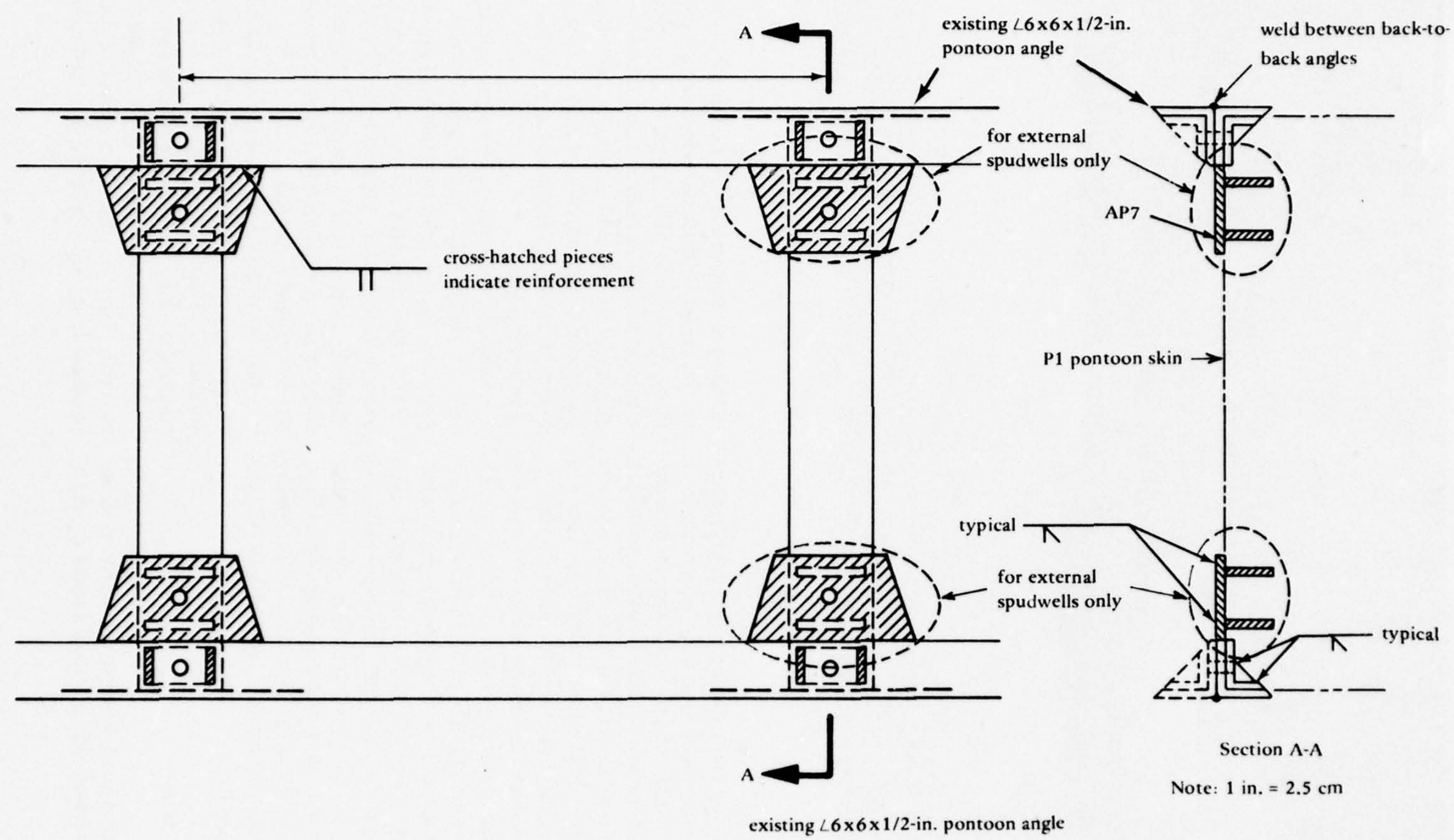


Figure 14. Structural reinforcements to increase shear strength of causeway.

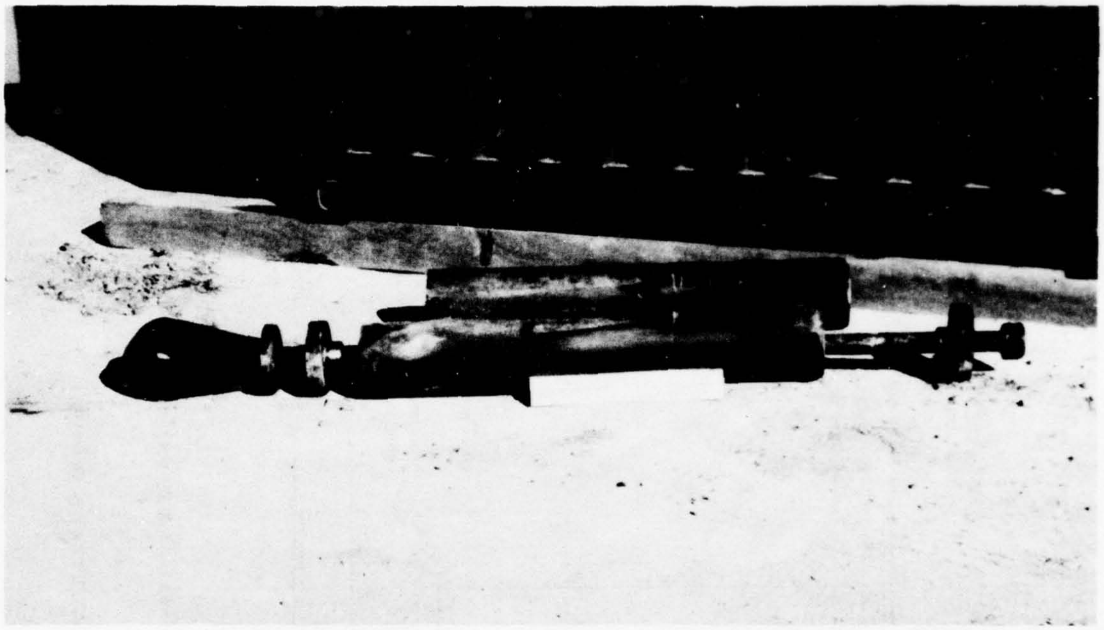


Figure 15. Components of shock absorbing lift system.

The lifting mechanism for the chain jacks features a shock absorbing system to reduce impact loads on the jacks as the causeway is lifted out of the surf. Figure 15 shows the shock absorbing system without the outside cylinder jacket. The system consists of a long lifting rod with an upper padeye that connects to the gimbals (load equalizer) for lifting. The rod, which is threaded at the bottom, passes through the center of a long rubber cylinder. A heavy washer, held by a large nut, caps the other end of the rubber cylinder. The entire system is mounted in a structural tube capped on the top and open on the bottom. The cap has a hole for the bar. When the upper padeye is lifted the lower washer compresses the rubber cylinder against the cap, creating a spring-type action. The steel tube confines the expansion of the rubber cylinders, thereby increasing the stiffness of the shock absorbing system.

2.2.3 Spudwell Tests

The development of internal and external spudwells for an elevated NL pontoon causeway was a problem in engineering design; however, some aspects of the design warranted field testing. The test program investigated the ability of the pile/spudwell complex to transfer moments and shears to the causeway structure. Internal spudwells were mounted in a pontoon framework for testing as shown in the Figure 1 insert. Members of the internal spudwell were strain-gaged.

The spudwell test results were extrapolated to approximate the design load criteria. Stresses measured in the spudwell members did not exceed 13,000 psi (91 MPa) when subjected to a pure moment-type load of 350,000 ft-lb (474.4 kN·m). No spudwell member stress exceeded 18,000 psi (126 MPa) when an exial load of 150,000 pounds (68,250 kg) was applied to the pile.

The spudwell design was constrained by the requirement to integrate a shock absorber system within the spudwell structure. The idea of using standard docking-type rubber fenders was improvised.

A short test program to determine the most suitable way to use the rubber cylinders was conducted. It was important for deflections to be

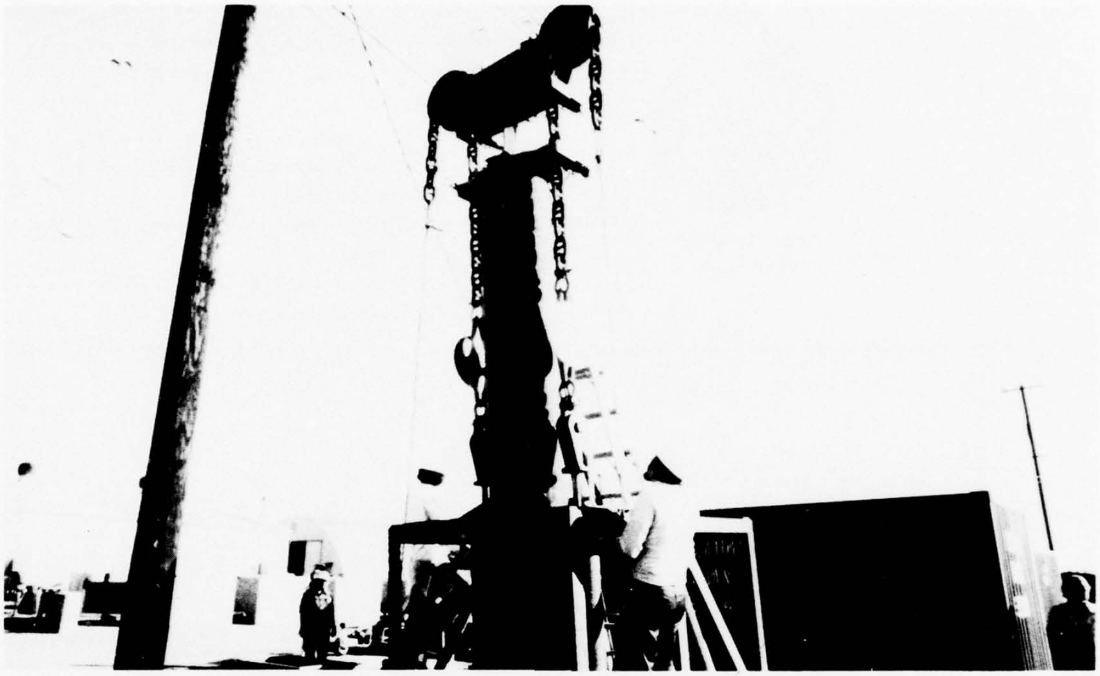


Figure 16. Load-deflection tests of shock absorber system.

limited to about 1 foot (0.305 m), and that the shock absorber system not be subject to sudden failure with the considerable energy stored in it. The solution was to use structural steel tubes to confine the rubber cylinders and to prevent the rubber from separating under load.

The basic stiffness parameter of the shock absorber unit is the ratio of the cross-sectional area of the rubber cylinders to the inside cross-sectional area of the confining tube. The test setup is shown in Figure 16. Loads were applied by deflecting the cylinders with one of the hydraulic chain jacks and measuring the loads with dynamometers. The test results are summarized by the load-deflection curves of Figure 17. The test data indicate that the rubber expands until it fills the void space when loaded in compression. After the void space is filled, the rubber stiffens greatly. Modifications to the rubber changed its load-deflection properties. For example, cutting the rubber into segments reduced the stiffness slightly, and drilling holes in the rubber modified the transition to the stiff portion of the curve. The drilled holes provided a deflection curve best suited to the shock absorber application.

During the Point Mugu tests one internal and one external spudwell was strain-gaged for gravity load tests. Maximum stresses on the internal spudwells occurred when loads were near the spudwell. The highest stress measured was 5,600 psi (39 MPa) when the crane was positioned with the rear wheels near the end of the causeway and the front wheels about 20 feet (6 m) from the end. The estimated live load on the spudwell at that time was 43,000 pounds (19,500 kg).

The external spudwell structure was less highly stressed than the internal, probably due to the more compact construction. The maximum live load on the spudwell due to the crane being positioned with its rear wheels on the cantilevered end of the causeway and its front wheels astride the spudwell was 51,400 pounds (23,300 kg). This load condition induced a maximum stress of 2,100 psi (15 MPa).

The spudwells performed the task for which they were designed. This is particularly true for the

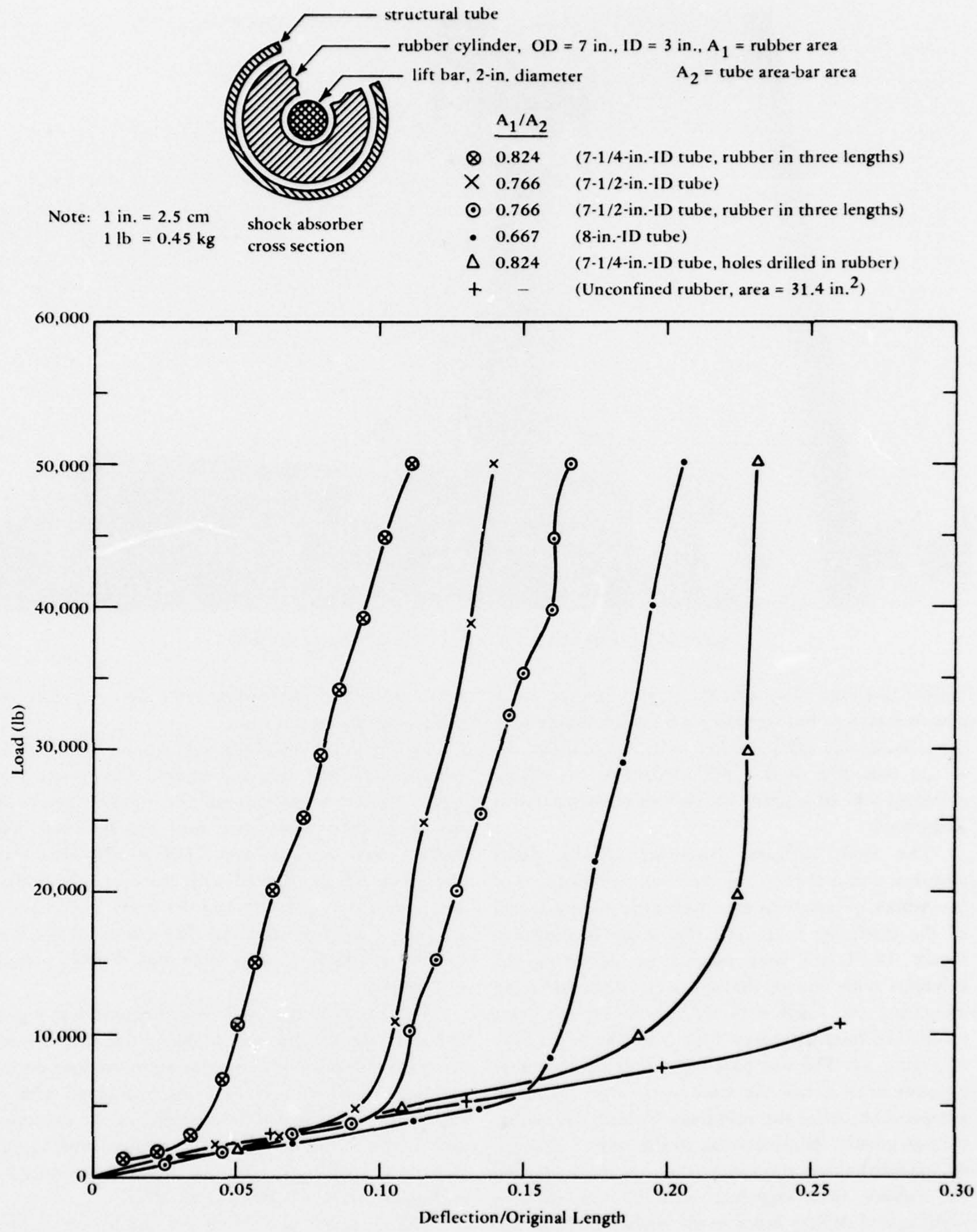


Figure 17. Load-deflection curves from tests of shock absorber configurations.

structural features. The measured strain gage data implied that the internal and external spudwells were capable of sustaining individual pile loads (working loads) of 170,000 pounds (91,000 kg) and 450,000 pounds (244,900 kg), respectively. The limiting pile/spudwell load is the spudwell-to-causeway connection or the pile-to-spudwell connection.

The limiting working load on the external spudwell-to-causeway connection is estimated to be 300,000 pounds (136,100 kg); however, the load transfer into the structural angle system is limited to about 180,000 pounds (81,600 kg). The internal spudwell capacity to transfer shear loads into the structural angle system is approximately 180,000 pounds (81,600 kg). The bracket-type pile-to-spudwell connection can be made sufficiently strong to transfer working loads by proper application of gusset plates.

The shock absorber system with rubber dock fenders worked quite well and will be retained in the spudwell system. The large lift padeyes which project above deck level prevent side-loading of causeways on LSTs; therefore, these padeyes will be made removable to provide a flush deck during transport.

The external spudwells on one side of each causeway section must be removed to be side-loaded on an LST, the basic causeway transport mode. Therefore, one half of the external spudwells must be installed after a side-carried causeway has been launched and is afloat. The installation of the test spudwells indicated that they would be very difficult to install at sea. The limiting factors are minimum tolerances and the requirement to insert bolts in the bottom angles which are underwater. The external spudwell attachment system will be modified to provide locking mechanisms that can be operated from deck level.

During the Coronado elevated causeway tests, one additional causeway section was added at the beach to provide more water depth at the crane platform. This section made a total of five sections from the beach to the pierhead. Time was not available to reinforce the section for external spudwells, and, therefore, it was used without modification. The spudwells performed satisfactorily under an estimated applied load of 110,000 pounds (49,900 kg).

2.3 SIDE CONNECTORS

The usual configuration of a floating or elevated causeway is a series of 21-foot (6.4-m) wide by 90-foot (27.4-m) long modular barges connected together end-to-end to create a long causeway extending seaward from the shore. However, new Logistics-Over-the-Shore (LOTS) concepts have requirements to expand the 21-foot (6.4-m) width to 42 feet (12.8 m) or wider to provide a large floating or elevated platform. The previous technique for side-to-side connection of causeways was to wrap chain around the structural angles of each causeway. This technique was acceptable for short-term operation in limited seas. A more substantial side-to-side connector is required to provide a secure connection for floating causeway barges. Two primary applications for the side connector are (1) to expand the elevated container off-loading platform and (2) to provide a large floating platform for unloading Roll-On/Roll-Off ships at sea.

2.3.1 Description

The design criterion for the side-connector device was to provide a stable transition between adjacent floating causeway sections so as to permit transit of vehicles up to a medium size crane [about 130,000 pounds (59,500 kg)] over the interface. In order to provide this stable platform, the adjacent causeway sections must be restrained from parallel and perpendicular translation. The structural connection between the sections could be designed as a fixed joint or as a hinged joint. A fixed joint provides a level transition between adjacent causeway sections, whereas a hinged joint allows rotation. A hinged joint was selected because a fixed joint generates greater moments and forces. Moreover, a fixed joint installation is likely to require underwater construction, which could be difficult and hazardous at sea.

The hinged joint was designed to integrate a strong, rugged joint with a practical field assembly. To accomplish this a rather simple mating-type connection was devised by utilizing the 9-inch (22.9-cm) wide gaps between individual pontoons of the causeways to mount the connection hardware.

Basically, the side connector, shown in Figure 1 insert, is engaged by bringing two causeway sections side-by-side, thrusting the male connector into the receiver slot on the adjacent causeway, and securing the connector fasteners.

2.3.2 Exploratory Tests

A requirement for a large floating platform was identified in 1973. Specific loading and operational criteria were not known. A field test was arranged with Amphibious Construction Battalion-ONE in early 1974 to investigate the parameters of making up a large floating platform.* Nine causeway sections were assembled in a 3x3 matrix. Two methods of holding the causeways side-to-side were used: (1) chains dogged off in the standard chain plates on each causeway section, as shown in Figure 18, and (2) long wire straps passed beneath the three sections and tied off to the outside section to prevent lateral separation of the causeways. In the longitudinal direction the causeways were connected end-to-end with standard end connectors.

A crane, a bulldozer, and a forklift were mounted on the assembled platform to determine whether equipment could drive on the platform. The causeways did not perform as an integral structural platform, but more like three individual causeway strings held side-by-side. This method has no horizontal or vertical shear resistance; therefore, causeway strings could displace vertically or horizontally between the sides. It would be difficult to drive across the causeway interfaces because relative vertical displacements exceeded 1 foot (0.3 m).

These exploratory tests demonstrated that a platform can be assembled at sea from individual causeway sections. The most significant finding from the tests was that side-to-side connected causeways must have a mechanism to prevent relative vertical and horizontal motions between sections.

After considering many side-connector configurations, a relatively simple concept using a structural beam between adjacent causeways was designed. Figure 19 shows the prototype side-connector design. Two of the units were built for testing in early 1975 in conjunction with Fleet operations with LASH barges. The units were used to side-connect a causeway section employed as a crane platform to unload LASH barges, as shown in Figure 20.

Conceptually, the side connector worked quite well, providing a solid transition for vehicle traffic across the joint between causeways. To hold the side-connected section in place, chains were engaged in existing chain plates. The chains allow 4 to 6 inches (10.2 to 15.2 cm) of in-and-out movement between the crane platform and the causeway. This movement proved to be detrimental to the side connector due to abrasive action between the steel beam and the steel receiver plates. After being moored at sea for 3 days, the steel plates were heavily worn.

Two important design criteria were revealed in the crane platform test. First, the basic plates and beams should be heavier to guard against abrasion. Second, the relative movement between the side-connected sections should be restricted to minimize abrasion. To restrict in-and-out movement requires that a method other than the existing chains and chain plates be used. The side connector shown in Figure 1 evolved from these criteria.

2.3.3 Tests with the Elevated Causeway

In the elevated causeway tests at Coronado, two sets of side connectors were installed to obtain the floating causeway/platform complex shown elevated in Figure 1. The floating platform was assembled in the harbor so that it could transport hardware to the erection site on Silver Strand Beach. The 100,000-pound (45,000-kg) crane was moved around to equalize causeway drafts for the side connectors. The first section was connected easily, but some difficulties occurred with the second side connection because the different types of end connectors between the paired sections have different tolerances.

After securing the turnbuckle assembly, the seven-section-long causeway, with two side-connected sections, was towed 15 miles (24 km) to the elevated causeway site by warping tugs. Upon reaching the open-beach site, the end of the causeway was beached with the floating platform offshore. The side-connected platform was just outside the surf zone and remained afloat for 12 days. Maximum wave conditions measured offshore were about 7 feet (2.1 m).

*CEL Special Report No. 55-74-05: "Floating causeway platform, truck turnaround tests," by B. R. Karrh, Apr 1974.

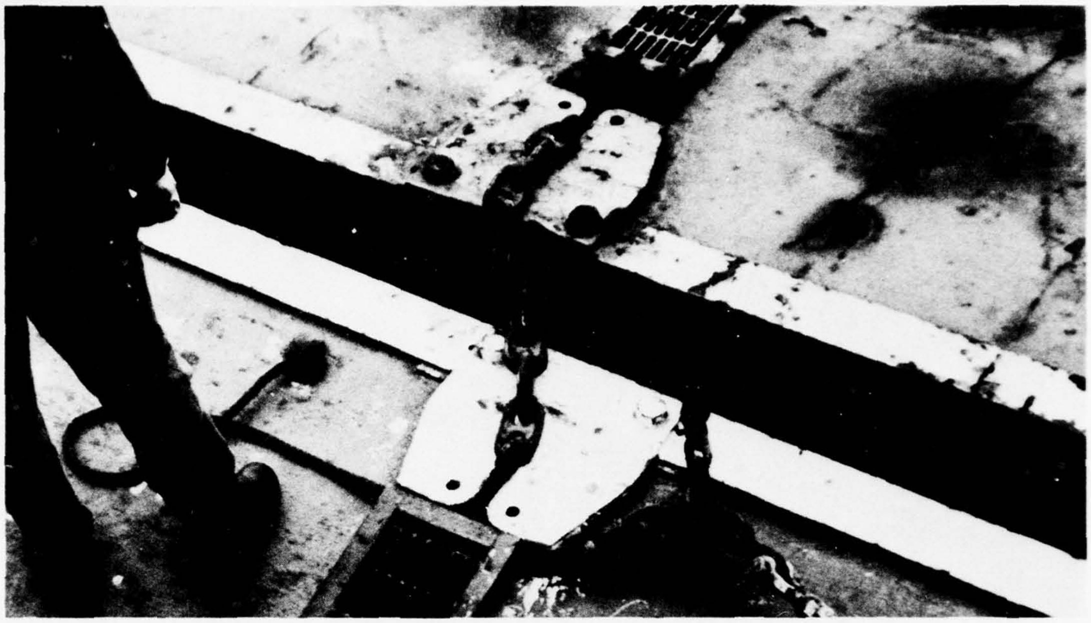


Figure 18. Chains used with standard causeway chain plates to hold causeways together. Note relative movement between sections.

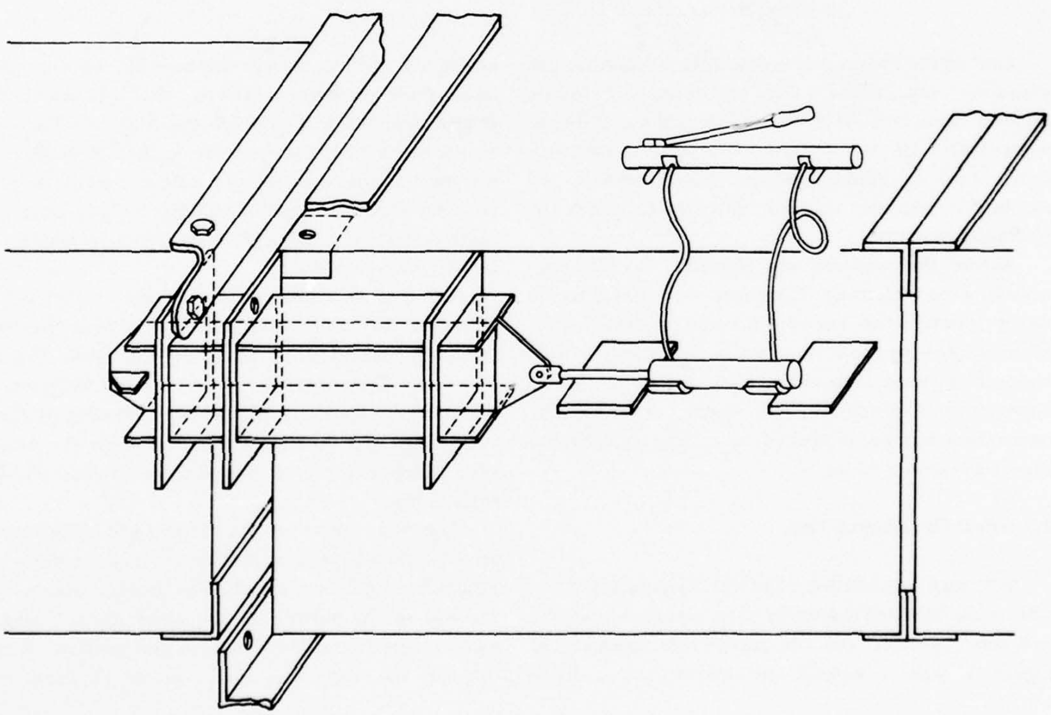


Figure 19. Sectional sketch of first semi-rigid side connector.

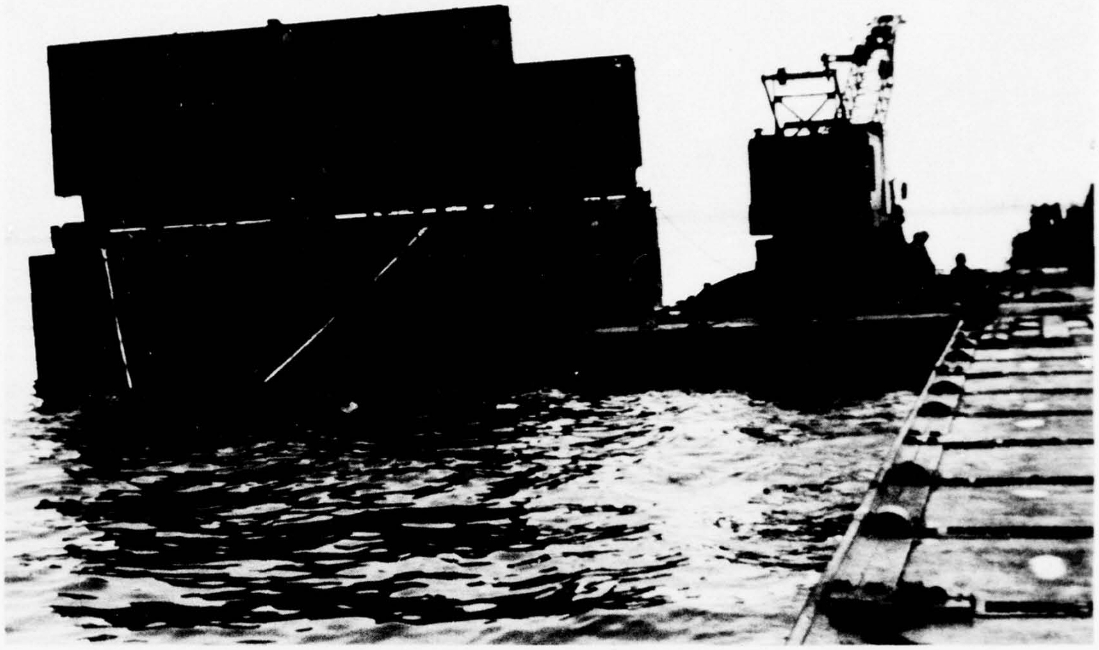


Figure 20. Crane mounted on side-connected causeway section during PHIBCB-ONE LASH barge operations, March 1975.

One of the Flexor end connectors failed while the causeway was afloat. The end-connector failure imposed additional load on the side connector due to wave forces on the causeway sections farther offshore. One hydraulic cylinder failed because the turnbuckle assembly was not tightened to relieve the hydraulic pressure.

Before the pierhead was elevated, the side connectors were withdrawn. Causeway motion due to the waves relieved the friction binding intermittently, thereby allowing the connectors to slide out. The side connectors were reinstalled to provide continuity between the elevated sections. Alignment for the side connection was accomplished by pulling the elevated causeways with a dozer.

2.3.4 Post-Operational Test

One recommendation of PHIBCB operators at the end of the elevated causeway tests was to replace the hydraulic cylinder and the turnbuckle assembly of Figure 1 with a simpler mechanism and a more accessible locking device, respectively. To accommodate these recommendations, the side-connection device depicted in Figure 21 was built and tested. A worm screw jack replaced the hydraulic cylinder as the mechanism to thrust the male connector into the receiver plates. A guillotine-type locking device at each end of the side-connector beam replaced the turnbuckle assembly.

The modified side connectors were tested at Port Hueneme in June 1976, using the four pierhead causeway sections to make a four-wide floating platform. Concurrently, a test was conducted to examine the feasiblity of making a floating platform with rigid side connections rather than the hinged type. Such a platform is required to unload RO/RO ships at sea.

The plan was to use the existing side connector as an intermediate step to bolting the angles together to establish a rigid connection. The problem was to bolt the legs of the bottom angles, which are submerged, using special tools developed for that purpose. A trial run of the test was made in the harbor. The

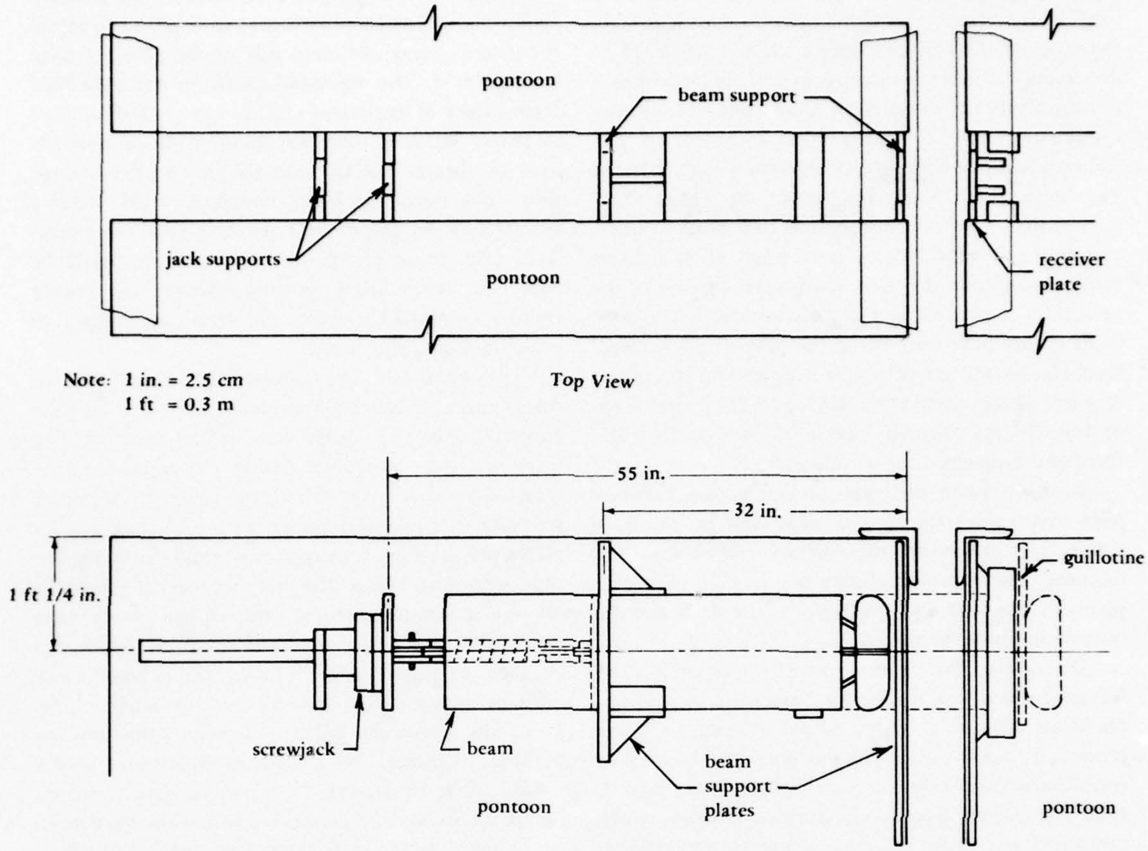


Figure 21. Side connector with screwjack activating mechanism.

preliminary test demonstrated that the bottom causeway angles cannot be bolted together with the special tools.

The four-wide causeway platform was assembled at sea with no major incidents using hinged side connectors. The screw jacks engaged and disengaged the side-connector beams using a powered drill to speed up the jacks. At times, the side connectors can be engaged at sea, but not in a harbor because causeway motions at sea relieve friction binding intermittently to allow the beams to slide into the receiver plates. The tests demonstrated that the screw jack and guillotine can functionally and operationally satisfy the side-connector requirements.

2.4 CAUSEWAY-TO-PILE CONNECTIONS

Loads and moments in the elevated causeway structure are transmitted to the piles and into the soil where the piles are driven. Design criteria for the spudwells were established at 150,000 pounds (68,000 kg) gravity load and 350,000 ft-lb (475 kN·m) moment. These loads originate from two independent sources. The vertical load comes from the dead load of the causeway plus live gravity loads, such as cranes, while the moments are generated by wave loads on the piles.

There are two reasons that a moment-resistant connection is preferred for connecting the piles to the

causeway. Figure 22(a) demonstrates the additional local load resistance of piles with the moment resistance over piles with hinged ends. Figure 22(b) illustrates the lateral stability gained by providing a moment-resistant connection at the pile-to-causeway connection. Safe operating criteria, based on pile strength and pile bearing capacity, for a single bent of an elevated causeway are shown in Figure 23.* Lateral load resistance is required to withstand wave, current, and wind forces plus loads from lighters moored alongside the pier. The greater rigidity of the causeway compared to the piles means that gravity loads induce little moment in the support piles; therefore, the assumption of simple supports for the causeway introduces small error. However, for lateral loads on the elevated causeway and local loads on the piles, fixed-end supports must be assumed.

Having chosen the type of connection between piles and causeway, it was necessary to design a connection to resist the vertical shear and the moment. The approach chosen was to weld triangular plates to the piles and spudwells at the deck and the bottom levels of the spudwells.

During the Phase I tests two CEL welders worked full time and two part time for 2 weeks to weld about 60 plates and other items. Actual working time was limited by wave splash on the piles and had to be coordinated with tides. Low strength rod was used to weld the plates, because it is more effective when corrosion and other foreign elements are present. Each plate required over 1 hour to install when no complications occurred.

Timely completion of the pile-to-causeway connections is an important practical aspect of the elevated causeway. Pile connections should not lag the elevating procedures by more than a few hours.

During the Phase II tests four civilian welders from Public Works Center and three welders from PHIBCB-ONE were employed full time. Figure 24 shows the welding equipment on the pier. Welding operations were interrupted when the elevating crew moved the equipment back and forth on the pier. CEL Drawing 75-24-1F of Appendix B details the plate installations for the Phase II tests.

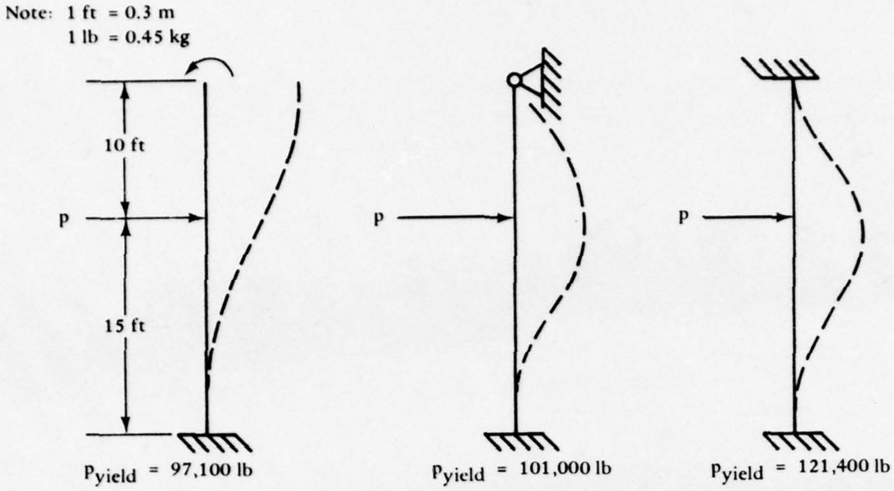
It was necessary to trim some of the plates with a cutting torch to fit squarely with piles that were driven out of plumb. There are two difficulties associated with the welded connection to the external spudwells. A welder's platform was required to install the plates on the outboard side of the piles as shown by Figure 25. The makeshift platform was assembled from pieces of angle and was clamped or tack welded in place. Welding time was limited on connections near the beach due to wave splash on piles at high tide. This same problem occurred while welding brackets during the Phase I tests, and surf conditions were even more severe. Extreme caution should be exercised when using cutting torches to remove welded components from the structural angles to avoid damaging the angles.

The Phase II tests provided a test bed to determine how well welders could keep pace with the elevating operation. Seven welders and seven welding machines were available during the tests. For the external spudwells on the access causeway sections, six plates per pile were required for a total of over 24 plates per section. These plates were all the triangular type shown in Figure 25. For the internal spudwells on the pierhead sections, the upper plates were welded directly to the pile and spudwell, which can be seen in Figure 1. The lower plates were more difficult because the welders had to climb down inside the spudwells to maneuver into position for welding the plates. Two of the lower plates required a welder of small stature to maneuver into position. Figure 26 shows a plate welded inside the spudwells.

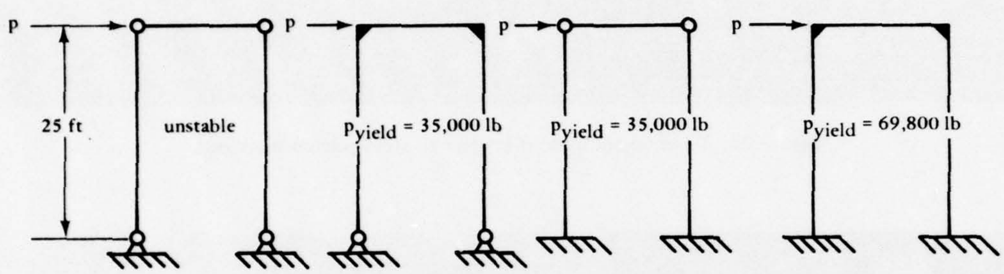
During the Phase II tests there were 270 plates welded to make up the pile-to-causeway connections. Discounting the welding for other parts of the test and other delays, five welders normally welded on the pile connections. The welding crew was on the job for about 100 hours, and about 50 percent of that time was applied toward productive welding of pile connections. Therefore, 350 man-hours were expended on 270 plates for an average of 1.3 hours per plate.

The welded connections require many PHIBCB steelworkers and much welding equipment. The size of the welding crew required is a function of the rate at which causeways are elevated. For example, if a causeway is elevated every 4 hours and one welder works on each available pile at a rate of one plate every 1.3 hours, eight welders are required to maintain the pace of the elevating operation.

*An analysis of lateral loads on the elevated causeway is being published under separate cover. The analysis includes gravity, environmental and operational loads on the causeway.



(a) Effect of pile end restraint on local-type loading.



(b) Effect of pile end restraint on lateral-type loading.

Figure 22. Effect of end conditions on pile resistance to lateral loads.

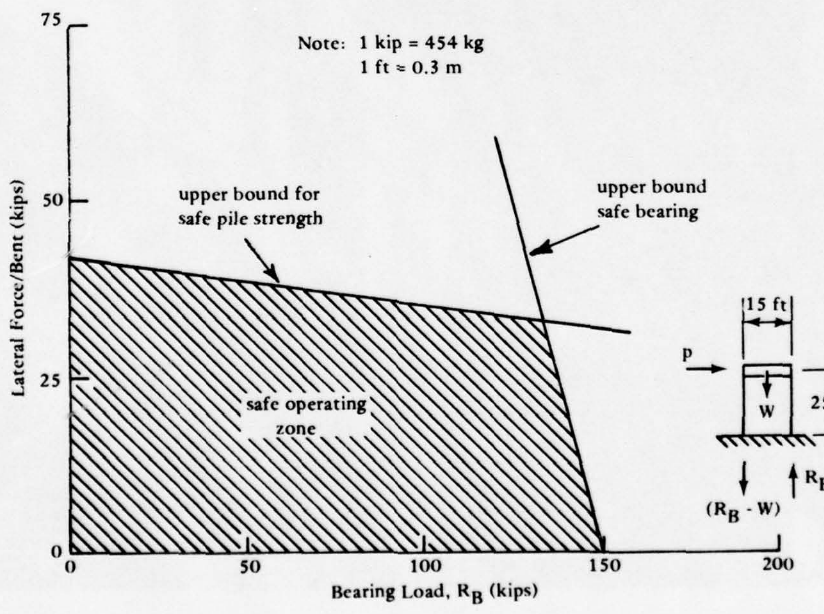


Figure 23. Safe operating criteria for combined lateral loads and bearing loads on one bent of the causeway.

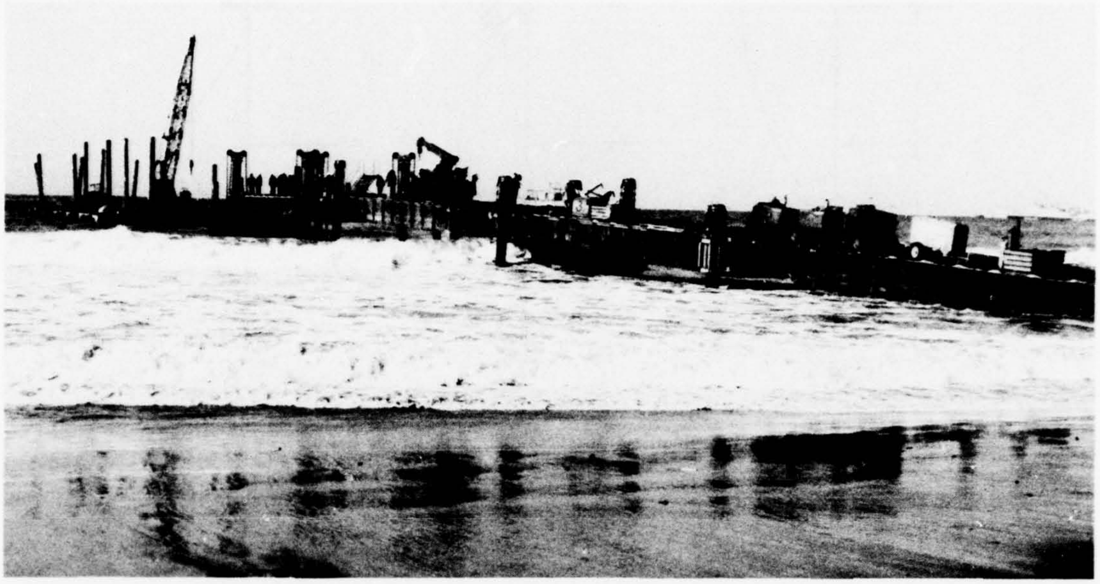


Figure 24. Welding equipment on pier to weld pile connections.

Figure 25. Welder on platform prepares to weld lower triangular bracket plates to pile for external spudwell. Note completed upper brackets.

Figure 26. Rectangular and triangular bracket plates used inside internal spudwells.

A causeway-to-pile connection that minimizes welding is needed to circumvent the steelworker requirement. Efforts were renewed to develop such a pile connection. One technique which employs steel bars inserted through holes burned through the pile walls is being tested. Some type of wedging mechanism is required to convert this technique to a moment-resistant connection. Other ideas that incorporate more sophisticated spudwell mechanisms are being investigated.* Many ideas seem to hinge around a two-part connection – a vertical shear mechanism plus a moment-resisting mechanism.

*The DeLong Corp., New York, New York, has a contract with CEL to study the pile connection problem.

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SECTION 3

STRUCTURAL ANALYSIS

As a floating structure, NL pontoon causeways demonstrated their capability to support many loads. The causeway will be called upon to support even heavier, more concentrated loads in carrying out the elevated causeway mission. The simple act of supporting the causeway on piles concentrates the loads into the support points, whereas there is relatively uniform support of the water pressure in the floating mode. Figure 27 emphasizes this load concentration. Greater moments and shears are introduced into the structure by the concentrated pile reactions. The basic objective was to define the structural response of the elevated causeway so that load capabilities and limitations can be predicted.

The optimum location of the causeway pile supports depends upon the interaction of dead and live load moments and shears as well as the length of the cantilevered end. Intuitively, the pile supports (and spudwells) should be located near the ends, the second or third pontoon being the most probable location. The results of an analysis to determine the optimum location of spudwells with respect to dead and live load stresses in the structure indicated that the optimum point lies between the second and third pontoons from the end. Since stress considerations provided no clearcut choice of spudwell locations, the second pontoon was selected so that potential structural problems would tend to be isolated between piles, and a six-pile-supported causeway could be used.

The objective of the structural investigation was to establish a reliable analysis technique that could be used to determine limiting load conditions and to enhance structural capacities by selective reinforcement. The methods by which the objective was accomplished proved to be diverse. Beam theory was used for the first estimate of structural capacity. Since beam theory does not account for the multiple discontinuities of the pontoon barge, a finite element model of the structure was developed for an existing computer code. The finite element model provided insight to the transfer of forces in the structure, particularly the behavior of the structural angles at the 9-inch (0.23-m) openings. As a result of the finite element study, a modified beam theory was developed that gives quick answers for maximum stress conditions. Local effects, such as load and shear transfers between pontoons and the structural angles, were examined. During the elevated causeway erection tests, two causeway sections – one supported by internal spudwells and one supported by external spudwells – were strain-gaged and tested with controlled loads.

The experimental and theoretical results are compared in Appendix A. The structural analysis was used to develop a prediction model for various types of gravity loads. Loads applied to the crane platform by the 90-ton (82-Mg) crane at Coronado are discussed, and support and causeway reinforcements are examined in Appendix A. Lateral load effects will be published at a later date.

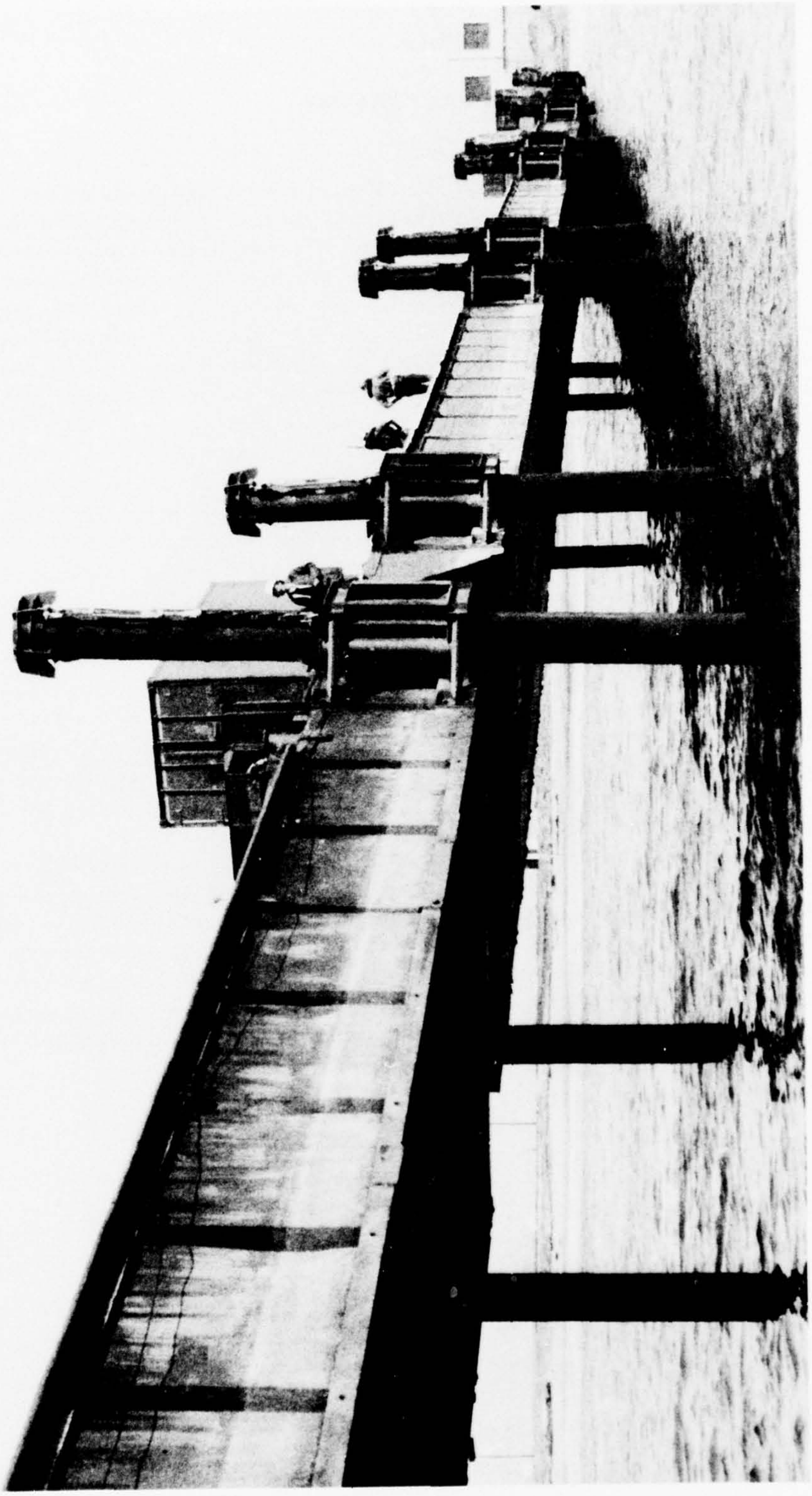

Figure 27. Pile supports concentrate loads into the causeway structure.

SECTION 4

SUMMARY

4.1 FINDINGS

1. Floating causeway sections in prior service 2 to 4 years could be used as elevated causeway approach sections after minor reinforcements.

2. External spudwells with bolt-on type attachment could not be installed on a floating causeway section at sea.

3. Stresses in the internal and external spudwells did not exceed 18,000 psi (126 MPa) when test loads were extrapolated to design loads of 350,000 ft-lb (474.4 kN·m) moment and 150,000 pounds (68,200 kg) axial load, independently.

4. Hinged-type side connectors with both hydraulic ram and screw jack activating mechanisms were installed in the 9-inch (0.23-m) openings between pontoons.

5. Hinged side connectors were used to assemble, at sea, four causeway sections into an 84-foot (25.4-m) wide by 90-foot (27.3-m) long floating platform.

6. Side-connected causeway sections were used as a floating logistics platform for a 30-ton mobile crane and in the erection of an elevated causeway pierhead.

7. The hydraulic cylinder activating device and the turnbuckle securing assembly of the side connector, though functionally adequate, were troublesome to PHIBCB operators.

8. A rigid side-to-side connection, made up by bolting adjacent angles together, could not be completed even with the use of special pontoon erection tools.

9. A moment-type pile-to-causeway connection increases the load capacity of the piles against local wave loads and increases the lateral (overturning) resistance by 100 percent compared to a hinged connection.

10. Each welded gusset plate of the pile-to-causeway connection required about 1.3 hours welding time to install.

11. Wheel loads to 16,000 pounds (7,260 kg) were safely supported by a bare pontoon deck. Dual wheel loads to 21,000 pounds (9,500 kg) were supported safely by a pontoon deck overlaid by 4x12 timbers. Higher wheel loads are possible provided the wheels roll directly on the structural angles.

12. Stresses in the structural angles of an elevated causeway were unpredictable by ordinary beam theory due to structural discontinuities at the 9-inch (0.23-m) opening between pontoons. A stress computation model must include terms to account for the transfer of shear across the openings.

13. A statistical comparison between stresses computed by the finite element model and the modified beam theory model determined that the average standard deviation of the modified beam theory model was about 40 percent less than the finite element model. Both models were compared to experimentally measured stress data.

14. Live crane loads up to 150,000 pounds (68,000 kg) were employed on elevated causeway structures instrumented by strain gages.

15. The safe stress level, as computed by modified beam theory, of four- and six-pile-supported causeways with 6-inch (15.2-cm) structural angles is reached by 130,000-pound (59,000-kg) and 200,000-pound (91,000-kg) wheel-mounted cranes, respectively.

16. Outrigger reactions to 150,000 pounds (68,100 kg) are developed by a P&H 9125 truck crane [140-ton (127,000-kg) rated] handling a 44,800-pound (20,900-kg) container at a radius of 50 feet (15.2 m).

4.2 CONCLUSIONS

1. The existing NL pontoon hardware configured in 3x15 sections, as modified and tested, is feasible and practical for the elevated causeway.

2. Pierhead sections require internal spudwells and a six-pile support. The sections must be reinforced to sustain the crane operating loads.

3. The approach to the pierhead can be of existing assault causeways adapted to accommodate external spudwells. Floating causeway sections that have been in service up to four years perform satisfactorily as elevated sections with external spudwells.

4. All piles should be 20 inches (0.51 m) in diameter with a 1/2-inch (12.7-mm) wall thickness.

5. The internal and external spudwell structures can safely transfer the design loads into the piles.

6. The external spudwell bolt attachment mechanism should be redesigned to permit installation at sea.*

7. A wide floating platform can be erected at sea from causeway sections fitted with hinged-type side connectors. The platform can be used for logistics operations or to assist in the erection of an elevated pierhead.

8. The hydraulic cylinder activating device and the turnbuckle securing assembly should be replaced with simpler mechanisms.*

9. A moment-resistant pile-to-causeway connection increases the load capacity of the causeway for a given size of pile.

10. The welded gusset pile-to-causeway connection requires more manpower and equipment than current PHIBCB staffing policy can provide.

11. A moment-type pile-to-causeway connection that minimizes field welding is needed.*

12. A bare pontoon deck can safely support a crane with a four-wheel bogie load up to 64,000 pounds (29,000 kg), or a six-wheel bogie load up to 96,000 pounds (43,600 kg). A pontoon deck overlaid by 4x12 timbers can safely support a crane with a four-wheel bogie load up to 84,000 pounds (38,000 kg), or a six-wheel bogie load up to 126,000 pounds (57,000 kg).

13. Modified beam theory is a more reliable method of computing elevated causeway stresses due to gravity loads than the finite element model.

14. Four- and six-pile-supported causeways with standard 6-inch (15.2-cm) angles should not be loaded with cranes exceeding 130,000 pounds (59,000 kg) and 200,000 pounds (91,000 kg), respectively.

15. Outrigger loads developed by a crane handling a 20-ton (18,200-kg) container at a radius of 50 feet (15.2 m) can be supported on an elevated causeway by (a) reinforcing the structural angles to effect a better shear transfer, (b) providing multiple pile supports near the outrigger locations, or (c) locating spudwells/piles such that the outriggers can be positioned directly on piles.

4.3 RECOMMENDATIONS

1. Use standard assault causeways, reinforced for external spudwells, to construct the elevated approach to the pierhead. The pierhead sections should be constructed with internal spudwells. All piles should be 20 inches (0.51 m) in diameter with 1/2-inch (12.7-mm) wall thickness.

2. Retain the basic external spudwell structure, but develop a new attachment mechanism so that external spudwells can be installed at sea.

3. Adopt the hinged-type side connector as a standard technique to connect causeways side-by-side.

4. Replace the hydraulic cylinder activating device and the turnbuckle securing assembly of the side connector with a worm screw jack and a guillotine-type locking mechanism, respectively.

5. Develop a moment-type pile-to-causeway connection that requires minimum field welding.

6. Each time a crane is moved on the elevated causeway, optimize wheel loads on the front and rear wheel bogies by rotating the boom or temporarily removing counterweights so that safe working loads are not exceeded.

7. Use modified beam theory to compute stresses in the structural angles of the elevated causeway. Use the influence lines for moment and shear (given in Appendix A) to expedite the computations.

*As of December 1976, the three hardware items of Conclusions 6, 8, and 11 appear to be resolved. The new hardware developments will be evaluated in the LOTS exercise during the summer of 1977.

8. Limit the four-pile-supported causeway with 6-inch (15.2-cm) angles to crane transit loads no greater than 130,000 pounds (59,000 kg). Limit the six-pile configuration to 200,000 pounds (91,000 kg).

9. Construct special causeway sections for the crane platform (pierhead) with internal spudwells located so that crane outriggers can transfer loads directly to piles.

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SECTION 5

ACKNOWLEDGMENTS

The following organizations provided direction, equipment, experience, and personnel necessary to achieve the excellent results, information, and satisfactory conclusion of the advanced development tests. Without their cooperation and support the program could not have been accomplished.

- Commander, Naval Surface Forces, Pacific authorized the Amphibious Units to support the program.

- Commander, Naval Beach Group, Amphibious Refresher Training Group, Coronado, approved and coordinated the beach support operations.

- Amphibious Construction Battalion One provided the personnel and equipment to direct, install, and operate the elevated causeway.

- Amphibious Assault Craft Unit One furnished the LCU landing craft and crews used to ferry the containers.

- First Force Service Regiment, First Marine Division, Camp Pendleton, California, provided the drivers and truck/trailers used to move the containers on the causeway.

- Naval Ship Research Development Center, Carderock, Maryland, conducted the motion measurements and analysis for the lighters moored to the pierhead.

- Naval Electronics Laboratory Center, Human Factors Division, San Diego, provided the human engineering study of the elevated causeway system.

- Public Works Center, U.S. Naval Station, San Diego, fabricated the spudwells, installed and load tested the external spudwells, and provided welders during the operation.

- Construction Equipment Department and Marine Terminal Division, NCBC, Port Hueneme, assembled all of the pierhead pontoon sections.

- Transportation Division, NCBC, Port Hueneme, provided operators and a construction crane for both the Phase I and Phase II tests.

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Appendix A

STRUCTURAL ANALYSIS

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THE FINITE ELEMENT MODEL

The NL pontoon causeways are made up of individual flotation modules interlocked by a system of structural angles. Multiple discontinuities between modules and the structural angle system clearly disqualify ordinary beam theory as the method for predicting stresses due to external loads. The finite element analysis method divides the engineering structure into many interconnected elements, or parts, capable of transmitting forces and displacements between each other. This technique was selected to analyze the causeway structure on piles.

The computer code chosen for the analysis was developed at the University of California at Berkeley. The code, entitled Structural Analysis Program (SAP) is capable of making linear, elastic analyses of complex three-dimensional structures subjected to static or dynamic loads. With the computer code available, the task of formulating an elemental mathematical model equivalent to the causeway structure on piles was undertaken.*

Constructing the Model

Model development was the most important step in the structural analysis. Figure A-1 schematically depicts the pontoon structure. From the standpoint of the analytical model, the causeway is an orderly collection of plate elements (pontoons) and structural angles, interconnected to function as an integral structure.

The angle members are the main members of the structure, with the pontoon boxes performing a supporting role. The top and bottom angles together can be thought of as the top and bottom flanges of a 5-foot (1.52-m) deep longitudinal plate girder with the boxes acting as the girder web. A problem with this analogy arises since the "web," which normally resists shear forces, is discontinuous. Where the web (or pontoon) is missing, the angles must transfer the shear across the gap, thereby inducing bending stresses. The problem is further complicated by the connection mechanism between adjacent pontoon strings.

A primary source of difficulty was to model the bolted connections so that the transfer of forces was consistent with the real structure. Figure A-2a and A-2b shows the two basic bolted connections, one exterior and one interior, between adjacent pontoon strings. Load transfer from the spudwells to the causeway structural system is most critical. For example, a gravity load placed directly on a pontoon must be transferred into the structural angle system. A simplified representation of this load transfer indicates that the upper connection attempts to transfer the load through the horizontal angle leg by bending, while the lower connection probably transfers the load directly into the throat of the angle from the pontoon corner. The lower connection seems stiffer than the upper one and may accept more of the load transfer.

The load transfer from pontoon to angle system is basically the same at the interior connection. Another problem occurs on how to transfer the load between adjacent pontoon strings. There are two paths through which the forces can travel. Figure A-1 and Figure A-2b show an AP1 plate bolted to both strings across the horizontal angle legs, and a bolt clamping the two vertical angle legs together. The inset of Figure A-2b depicts the distortion that occurs to the AP1 plate due to the bending-type loading it receives. The bolt that clamps the vertical legs together effects a direct shear transfer and is considerably stiffer than the AP1 plate; therefore, it probably accepts more of the shear transfer between strings. The AP1 plate functions as the resisting element for transverse moments in the structure. In reality the interior connection is probably made up of two relatively independent components – the vertical shear resisting bolts in the vertical angles legs and the transverse-moment-resisting AP1 plates. The qualitative analysis

*Mr. John E. Crawford, computer/structures specialist at CEL, developed the elevated causeway finite element model.

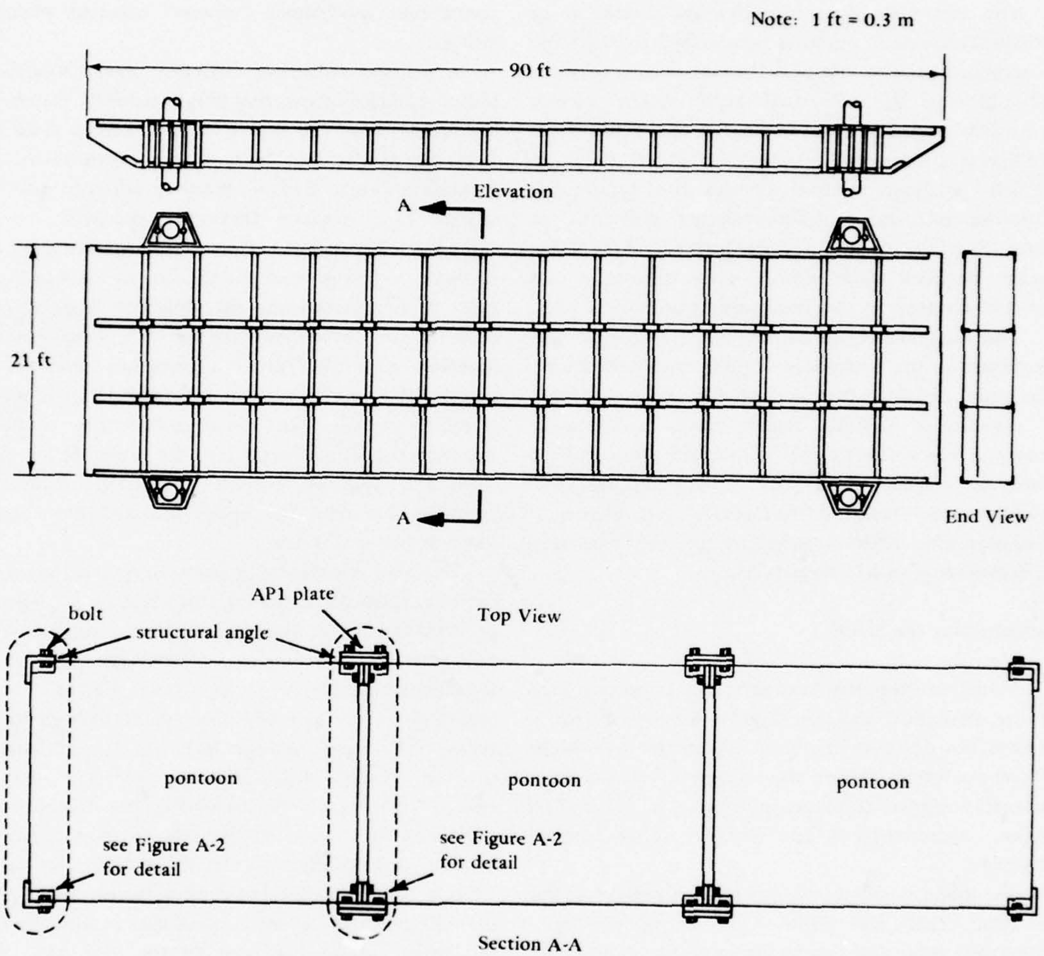


Figure A-1. Schematic of 3x15 pontoon causeway structure.

above indicates the pontoon connections to be structurally inefficient for transfer of gravity loads.

The finite element causeway model is represented in Figure A-3. The model is made up of beam elements and plate elements with the two types of elements connected at nodes representing bolted connections. Each outside angle becomes a beam, and each beam is subdivided into beam elements. The back-to-back interior angles are represented by a single beam with the properties of the two angles. Beam nodes* correspond to the actual places where the pontoons bolt to the angles. Each pontoon model is made up of six plate elements with the corners of the top and bottom plates attached to the beams at the nodes. In a simplified manner, the causeway model is made up of eight beams (the angles), which are tied into a structure by the plate elements (the pontoons). The external spudwells were modeled as a collection of beam elements attached to the causeway and to the supporting piles.

Stress predictions in the real structure can be no better than the model representation. In the case of the causeway structure model, the objective was to analyze the primary forces and stresses in the real structure. Secondary behavior of the structure, such as connections, was simplified to limit the number of elements and nodes to a manageable level. With the selected modeling, there were more than 200 beam elements and nearly 300 plate elements. The connection mechanism was studied in more detail using a substructure model.

Results of the Model Study

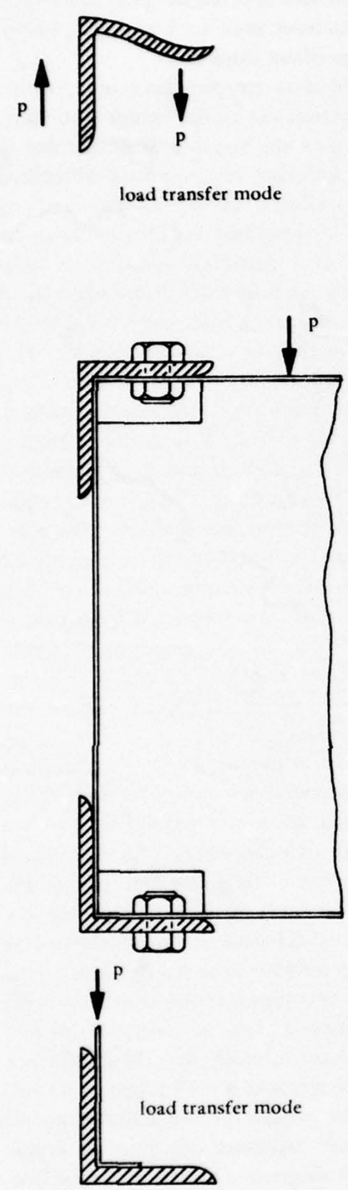
The objective of the finite element study was to develop a mathematical model to predict the structural behavior of the complex and highly discontinuous, modular causeway structure. The model was successful in this respect because it revealed stress variations and the response of the structure to different loads. Results provided by the finite element model led to the development of the modified beam theory model. However, the finite element method as well as this particular model are not well suited to the analysis of the local stress caused by the rather complicated loadings applied (e.g., portable cranes plus outriggers). This type of analysis was needed to compare with experimental data and to compute the maximum stresses from different loadings. Also, it should be noted that the finite element technique is better adapted to analyses of continuous structures than to a modular, bolted structure like the pontoon causeway.

Load and stress data are presented in a manner that will be of practical use to the reader who wants to estimate stresses in the elevated causeway due to specific loads. An influence line is a classical method for showing the effects of a moving load on structures, such as bridges, but it is usually drawn for moment or shear at a particular location or cross section. By inserting the properties of the structure at that cross section, influence lines can be drawn for stress at a cross section in a similar manner. This approach was used to depict the stress at a point due to a unit load at any point on the causeway structure.

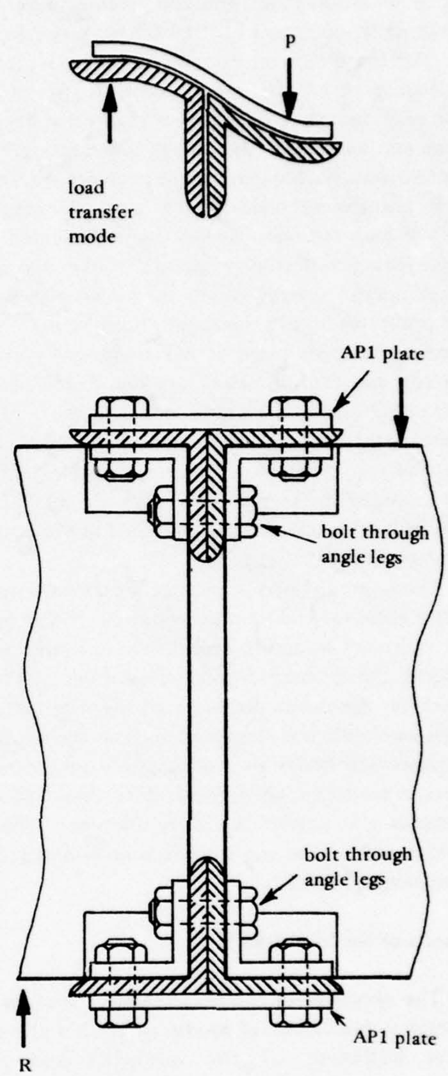
Influence lines for unit loads were drawn for the intersection between the third and fourth pontoon from the end of the causeway, which is the most highly stressed cross section; these influence lines are presented in Figure A-4. Two support conditions are given: (1) the 3x15 causeway supported by four piles, and (2) the 3x15 causeway supported by six piles. The influence lines for the four-pile-support condition are presented for 6-inch (15.5-cm) and 8-inch (20.3-cm) angles. Six-inch (15.5-cm) angles are standard for all causeways.

The concept of influence lines is sometimes confusing to persons who are unfamiliar with them. Consider an example of a crane weighing 100,000 pounds (45,400 kg) with the wheel loads distributed as shown on Figure A-4. Read the ordinate of the influence diagram at each load, and multiply the ordinate times the wheel load at the corresponding location. Table A-1 compares the stresses at x = 19.5 feet (5.94 m) for the support and structural angle conditions indicated for a 100,000-pound (45,400-kg) crane. Note carefully that these influence lines apply only for stress at x = 19.5 feet (5.94 m). Stresses due to the weight of the 130,000-pound (59,000-kg) causeway structure are given in Figure A-5. This is a stress diagram – not an influence line.

*A node is where two or more elements intersect.



(a) Exterior pontoon-to-angle connection.



(b) Interior connection between strings.

Figure A-2. Exterior and interior connections between pontoons and angles and pontoon strings.

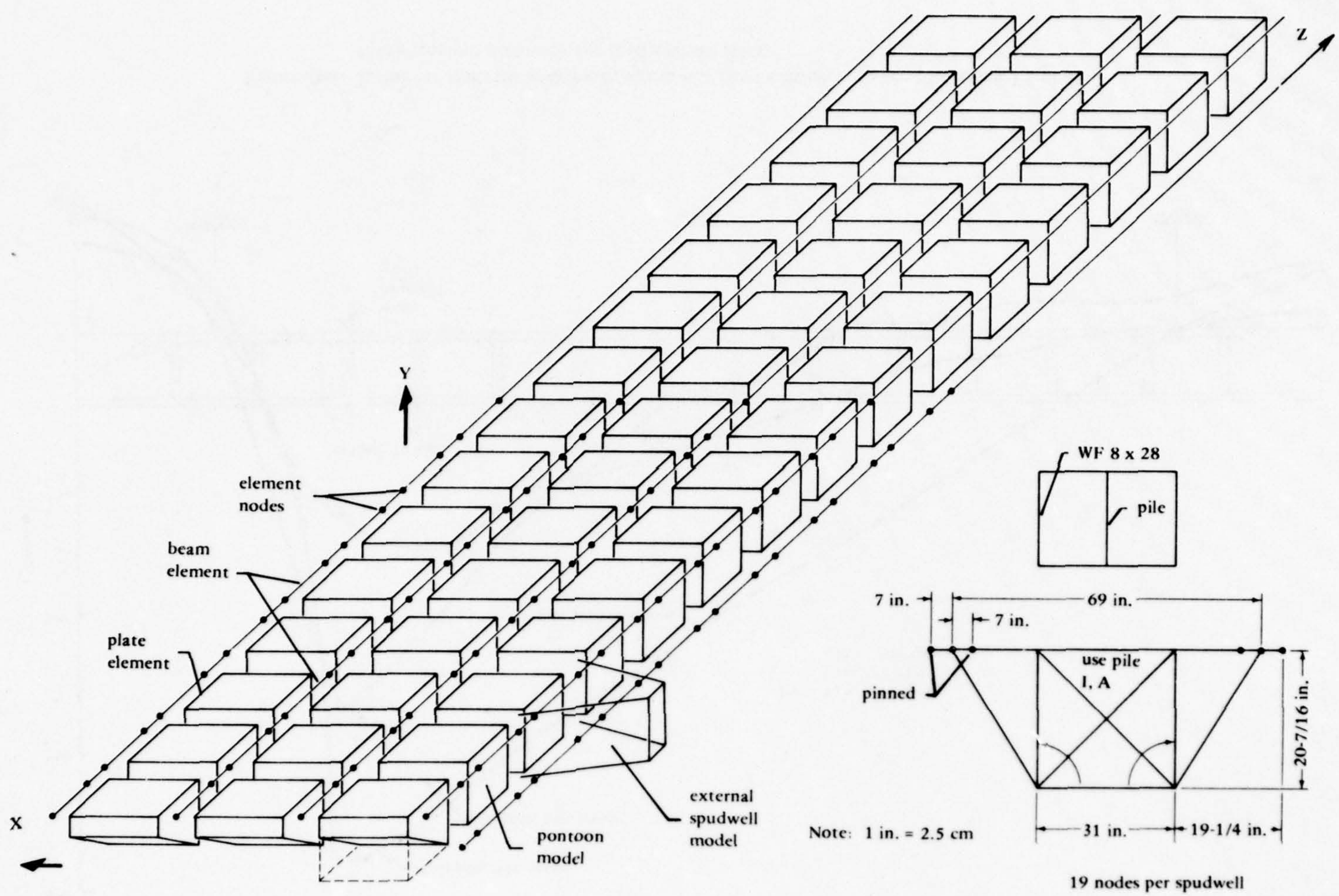


Figure A-3. Finite element model of a 3x15 pontoon causeway with external spudwells.

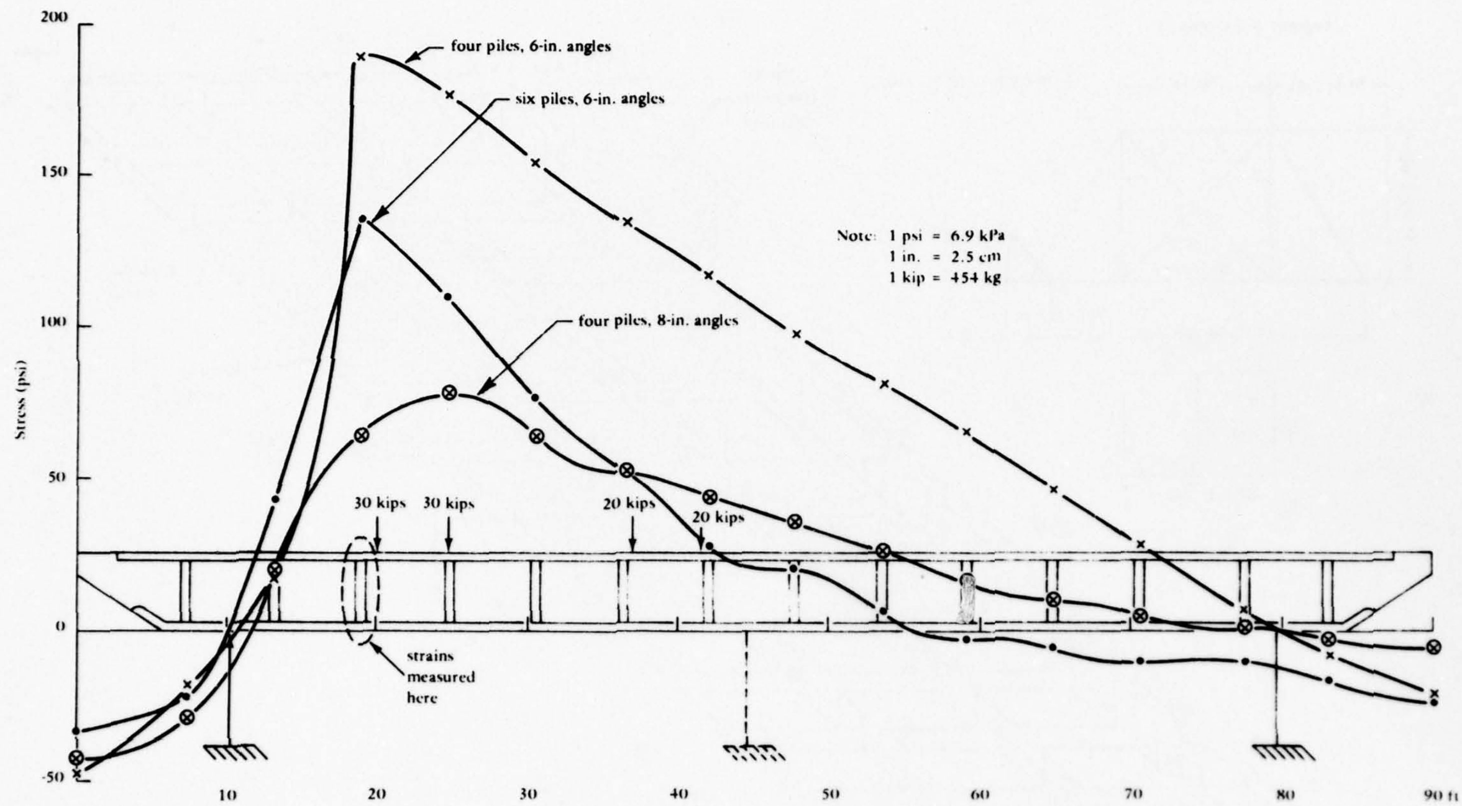


Figure A-4. Influence lines for maximum stress at a cross section between pontoons 3 and 4 – finite element solutions for four and six piles.

Table A-1. Comparison of Critical Stresses from 100,000-Pound (45,400-kg) Crane

Condition	Critical Stress (psi) With –				Gravity Load (psi)	Total Stress (psi)
	30-kip Wheel at 20.0 feet	30-kip Wheel at 24.5 feet	20-kip Wheel at 37.0 feet	20-kip Wheel at 41.5 feet		
Four piles, 6-inch angles	5,100	4,650	2,300	2,040	7,500	21,600
Four piles, 8-inch angles	2,130	2,370	1,040	880	3,600	10,020
Six piles, 6-inch angles	3,960	3,300	1,020	620	2,180	11,080

Note: 1 psi = 6.895 kPa
1 in. = 25.4 mm
1 ft = 0.3048 m
1 kip = 454 kg

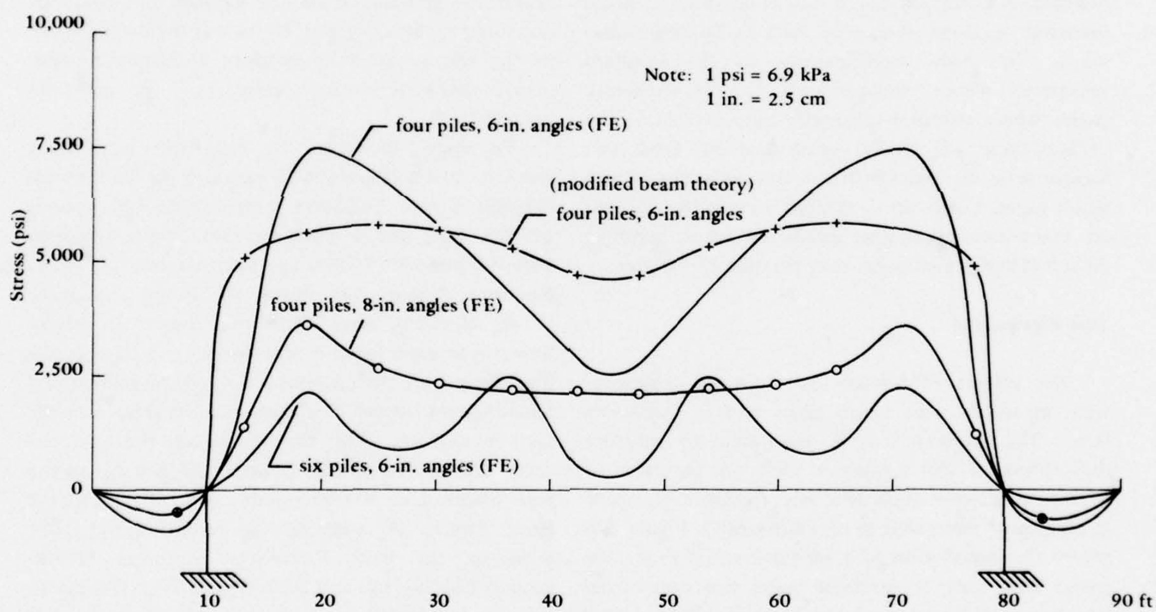


Figure A-5. Stresses in four- or six-pile elevated structure due to causeway weight.

STRUCTURAL TESTS

Load tests of the causeway structure were conducted concurrently with the development tests of the causeway jack-up system. During the Phase I tests at Point Mugu, two causeway sections were selected for strain gaging – one section with internal spudwell supports and one section with external spudwell supports. Test loads were simulated on the finite element computer model to develop stress data to guide the placement of strain gages on the causeway structures. Correlation of the test results with analytical methods can determine the accuracy of the analytical models.

During the Phase II tests at Coronado, California, limited strain gage testing was conducted. The primary objective of monitoring stresses was to check critical stress points. The transition length of the causeway was subjected to a 150,000-pound (68,000-kg) crane compared to a 100,000-pound (45,400-kg) crane in the Phase I tests. The entire five-section transition length was made up of standard causeway sections previously used as floating causeways. The only modifications to the standard causeways were reinforcements to the causeway angles where external spudwells were attached. The effectiveness of special reinforcements and pile locations on the crane platform was measured by two strain gages. Loads up to 90,000 pounds (40,800 kg) on crane outriggers were predicted when handling 20-ton (18-Mg) containers over the side of the pier.

Test Equipment

The effects of known loads on the causeways were measured with strain gages and an engineer's level. The engineer's level was used to measure deflections of the causeway with an accuracy of 1/100 of a foot (0.00305 m). Weldable electrical strain gages* were used to measure strains. Figure A-6 shows the installation of a weldable strain gage. The initial high cost of weldable gages was more than compensated for by the labor and time saved during installation, plus the additional reliability factor of a factory-sealed gage.

Thirty-three gages were installed on the structural angles of the two selected causeway sections at the Phase I tests. A direct-reading Baldwin-Lima-Hamilton strain indicator with a manual switching unit was used to monitor load-induced strains. The measurement circuitry consisted of a quarter-bridge, three-wire arrangement with the compensating gage at the indicator. Strains were read and recorded manually for each load condition.

Three strain gages were applied to the causeway sections for the Phase II tests. Strains were recorded through a magnetic tape data monitoring system. Continuous strain output was transcribed on a strip chart for data reduction. Strip chart data were available for immediate analysis at the test site.

Description of Tests

The Phase I test program was designed to compare stresses calculated analytically with stresses (from strains) measured in the field. Test loads were chosen to generate causeway stresses that could be measured by strain gages. It was not within the scope of the test program to conduct a complete stress-strain measurement analysis of the pontoon causeway.

To apply loads to the causeway sections, a concrete block (Figure A-7) weighing 30,000 pounds (13,600 kg), a bulldozer weighing 66,800 pounds (45,400 kg), and a wheel-mounted crane weighing 100,000 pounds (45,400 kg) were placed at different locations. Figure A-8 shows the dozer and crane during elevating tests. With the center of gravity known for each load, it was possible to control the load "seen" by the causeway through placement and blocking procedures. The crane was the most versatile load mechanism. With the boom over the rear, the crane distributed 60,000 pounds (27,200 kg) to the rear wheels and 40,000 pounds (18,100 kg) to the front wheels. By rotating the boom to the front position, the load distribution becomes 80,000 pounds (36,300 kg) and 20,000 pounds (9,100 kg) to the rear and front wheels, respectively. Jacked up on outriggers, the rear outriggers take 31,000 pounds (14,100 kg) and the front, 69,000 pounds (31,300

*Weldable gages can be completely installed in 15 to 30 minutes. The commercially available gages come pre-wired and waterproofed, and require only that the lead wires be distributed and connected into strain-recording equipment. Field installation of weldable gages requires only a surface grinder, a small portable generator, and a capacitive discharge spot welder.

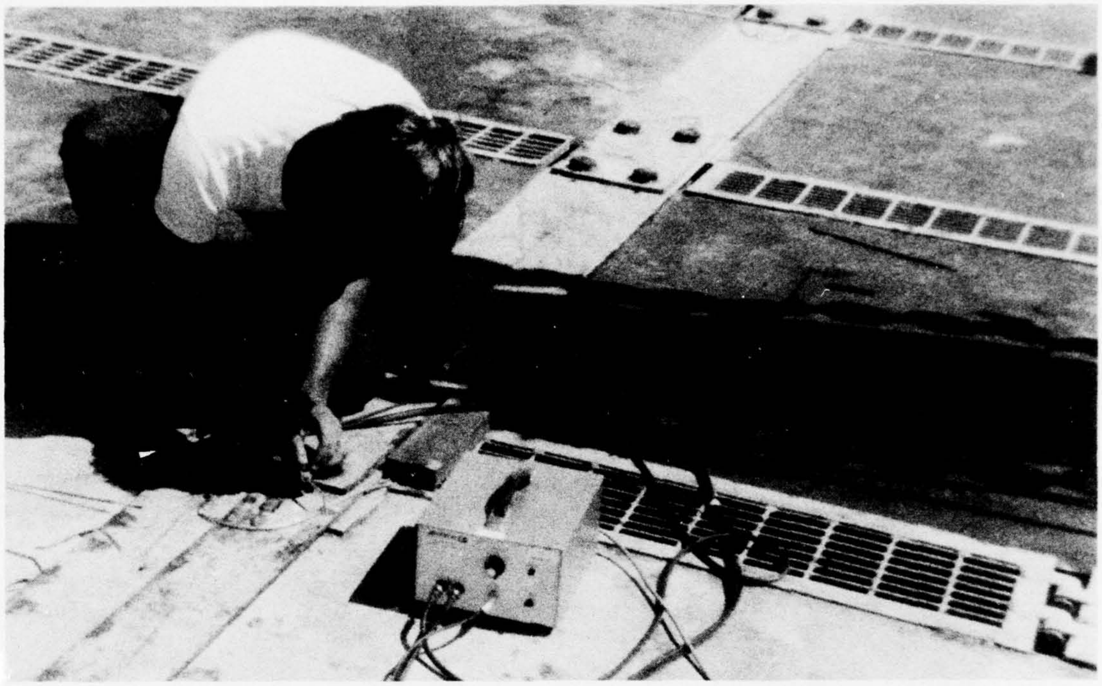


Figure A-6. Field installation of weldable strain gage.

kg). Each causeway section was loaded with eight block positions, four dozer positions, and eight crane positions. The crane was moved from end to end of the causeway to measure the variation of stress at a point. Selected loadings are designated in Table A-2.

One causeway section supported by external spudwells and one by internal spudwells were strain-gaged so that strains from similar loads could be compared. Strain gages were applied to each section in geometrically corresponding locations (Figure A-9). Also, critical members of the internal and external spudwells were strain-gaged.

For the Coronado tests, one gage was installed on the causeway nearest the beach and two gages on the crane platform. The section near the beach was instrumented between the third and fourth pontoons from the end. The strain gages on the crane platform were installed to measure the effectiveness of the crane reinforcement. One gage was applied to the reinforced angle and the other adjacent to the internal spudwell subjected to the greatest loads.

Few formal tests were conducted at Coronado. The first mounting of the 100,000-pound (45,400-kg) crane on the elevated causeway was monitored, and the results were examined to ascertain the safety of driving the 150,000-pound (68,000-kg) crane onto the causeway. Preliminary results indicated that it was safe to proceed. Twenty-six thousand pounds (11,800 kg) of counterweight were removed when the big crane first drove onto the causeway, making the weight 124,000 pounds (56,200 kg). An interesting sidelight developed during the tests. The beached section was partly underwater at extreme high tide, and the variation of wave pressure on the bottom of the causeway caused significant stresses in the causeway angles.

The gages on the crane platform were monitored as the big crane maneuvered into position for container off-loading and as it handled 20-ton (18-Mg) containers. As a special test, the crane was rotated 360 degrees at 90-degree increments. This test was conducted with no load and with a 20-ton (18-Mg) container on the hook. The 26,000-pound (11,800-kg) counterweight was not removed for the crane's departure.

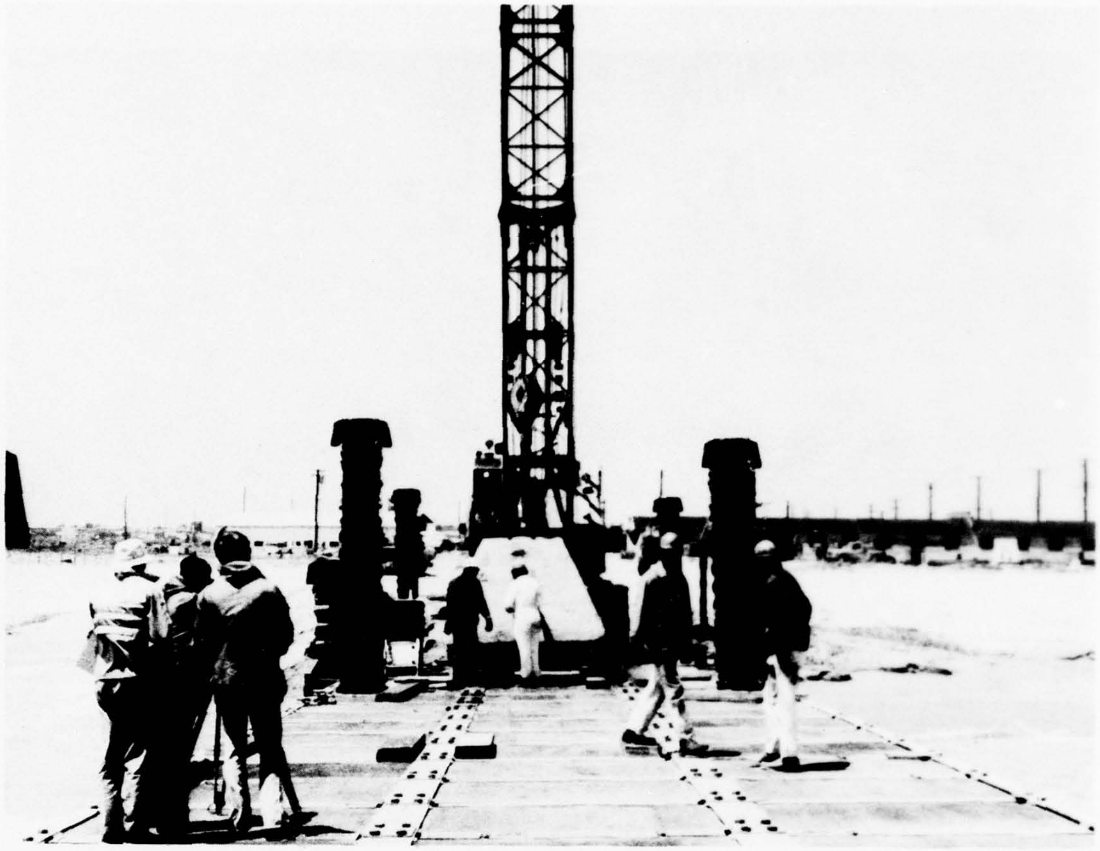


Figure A-7. Concrete block weighing 30,000 pounds (13,600 kg) used to load causeway.

Figure A-8. A bulldozer and crane, weighing 66,800 and 100,000 pounds (30,480 and 45,630 kg), respectively, were used to load the strain-gaged causeway structure.

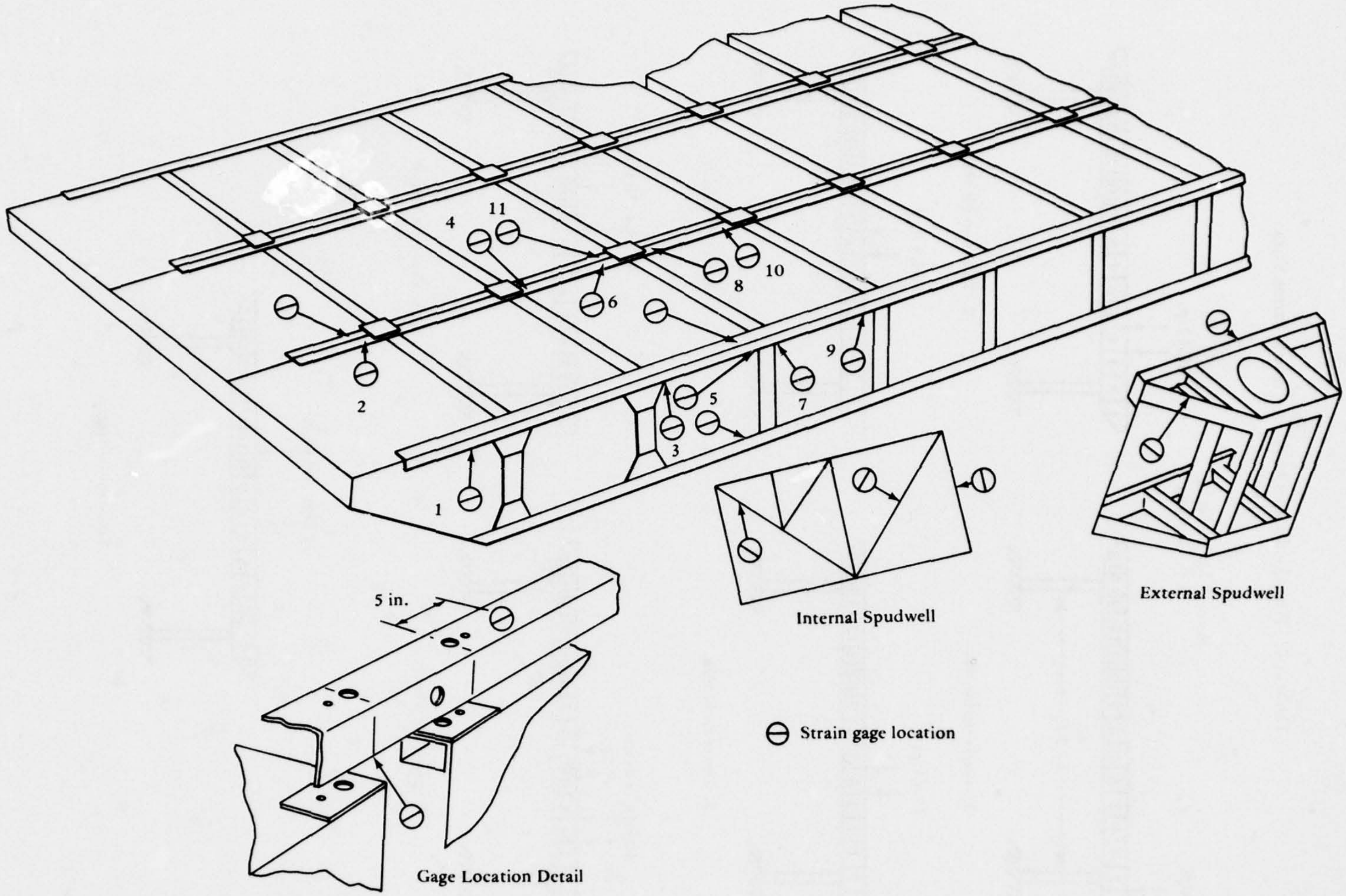
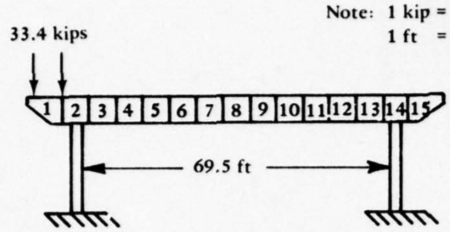


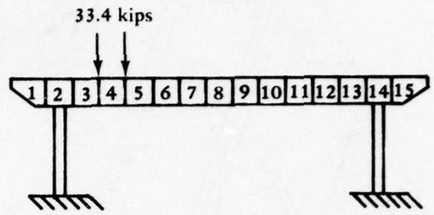
Figure A-9. Strain gage locations for causeways with internal and external spudwells.

Table A-2. Test Load Cases Used to Present Stress Data

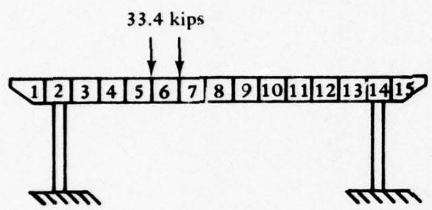
Note: 1 kip = 454 kg
1 ft = 0.3 m



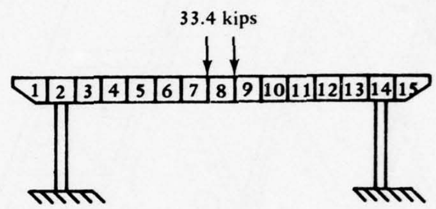
1. Dozer on blocks.



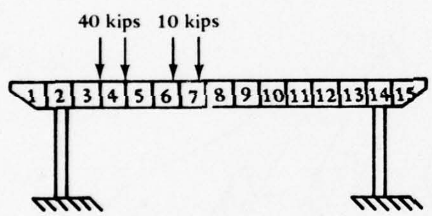
2. Dozer on blocks.



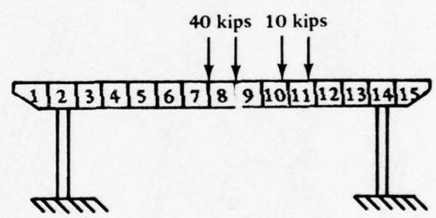
3. Dozer on blocks.



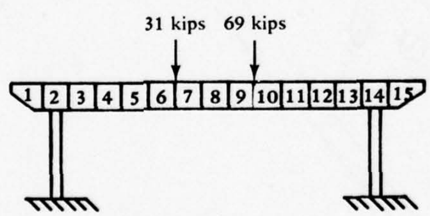
4. Dozer on blocks.



5. Crane on wheels.



6. Crane on wheels.



7. Crane on outriggers.

Test Results

Table A-3 is a summary of stresses at seven selected locations on the causeway structure for seven different loadings. These locations were chosen as being the most representative of the gaged locations. The table presents stresses for the same point derived from four independent sources – two experimental and two theoretical – to permit direct comparison between the methods. The theoretical stresses were computed by the finite element model and by beam theory modified to account for shear stresses across the openings between pontoons.

It may be seen that the stresses compare reasonably well, with only a few isolated instances where significant differences occur. For example, the finite element model stresses differ considerably from the experimental and modified beam theory stresses where strain gage seven is located. In general, the modified beam theory seems to follow the trends of the experimental data better than the finite element model.

A statistical comparison between pairs of stresses was made. The approach taken was to compare the differences between stresses derived from two techniques. For example, strain gage 5 for load case 4 reads -4,080 psi (-28,131 kPa) for external-spudwell-supported tests and -4,514 psi (-31,124 kPa) calculated by modified beam theory. The difference is 434 psi (2,992 kPa). By accumulating such differences for each load case and performing some basic statistical mathematics, Table A-4 was developed. The standard deviation was computed according to the following formula:

$$\text{Standard Deviation} = \left[\sum_{i=1}^{n} \frac{(\text{Stress } 2_i - \text{Stress } 1_i)^2}{\text{no. of measurements, } n} \right]^{1/2}$$

Average differences were computed as follows:

$$\text{Average Difference} = \sum_{i=1}^{n} \frac{(\text{Stress } 2_i - \text{Stress } 1_i)}{\text{no. of measurements, } n}$$

The median is simply the middle value of all the seven differences considered.

The statistical meaning of the standard deviation says that 68 percent of the measurements lie within one standard deviation; therefore, the smaller the number, the closer the comparison. When comparing the theoretical results with experimental data, one assumes the experimental data to be reasonably valid. There is a significant degree of consistency to the experimental stress data, and the statistical analysis indicates a clear advantage for the modified beam theory.

Table A-4 is divided between load cases to indicate how well stresses correlate within each load pattern. Interestingly, the greatest variations occur where the load is applied in the vicinity of the strain gages, as in load cases 2 and 5. Apparently local stresses, which are not considered in the theoretical methods, are responsible for this condition.

The measured stress variation at the opening between the fourth and fifth pontoons as the *100,000-pound (45,400-kg) crane moved from one* end to the other end of the causeway with external spudwells is shown in Figure A-10. For comparative purposes the computer model and the modified beam theory stresses are shown on the same plot. In effect, this graph can be interpreted as an influence line for stress, using a crane load instead of a unit load. Figure A-11 shows a similar plot of the maximum stress between pontoons 3 and 4 for a 150,000-pound (68,000-kg) crane measured during the Phase II tests. The failure criterion (defined later) indicates a safety factor of 2.8 for this loading based on maximum measured stresses.

Stresses along the length of a causeway angle due to a 100,000-pound (48,500-kg) crane near midspan are shown in Figure A-12. Note the extreme variation of stresses over the openings between pontoons. Vertical shears are responsible for this effect. The finite element stresses fluctuate more than the modified beam theory stresses, but the measured and computed stresses exhibit similar patterns, thereby verifying the structural behavior. Table A-5 compares the measured stresses in the back-to-back causeway angles for different loads. Both theoretical models assume equal stresses in these angles.

Deflections of the causeway structure loaded by the crane were measured with an engineer's level. A grid of deflection was read to determine deflections across as well as lengthwise of the causeway. The engineer's level lacked the precision to detect the small deflections across the causeway, but the lengthwise deflections due to the 100,000-pound

Table A-3. Summary of Stresses for Dozer and Crane Loadings – Experimental and Theoretical

Load Case[a]	Data Source[b]	Stress (psi)[c] at Strain Gage[d] –						
		4	5	6	7	8	9	10
1	Exp (IS)	1,920	-420	2,460	1,620	1,020	0	1,410
	Exp (ES)	3,240	–	1,470	1,960	1,350	0	1,590
	Theo – FE (ES)	1,939	-939	1,319	1,438	920	375	1,141
	Theo – MBT	1,864	1,041	2,184	2,213	1,399	1,731	1,545
2	Exp (IS)	-3,270	-5,700	0	2,550	-1,950	-2,760	-690
	Exp (ES)	-1,410	-5,280	120	2,910	-2,250	-2,520	-1,660
	Theo – FE (ES)	-1,072	-4,085	811	1,317	-3,801	-354	-1,510
	Theo – MBT	-3,314	-8,238	-103	3,475	-4,660	-1,988	-886
3	Exp (IS)	-2,190	-5,100	1,290	3,060	-3,900	-5,940	-330
	Exp (ES)	-1,680	-5,310	570	1,940	-2,910	-6,630	-600
	Theo – FE (ES)	-826	-4,971	1,785	304	-3,310	-2,843	1,572
	Theo – MBT	-2,636	-6,252	287	2,193	-3,385	-7,743	-1,210
4	Exp (IS)	-1,410	-3,030	810	3,390	-2,760	-3,870	-240
	Exp (ES)	-1,080	-4,080	420	2,420	-2,190	-5,100	0
	Theo – FE (ES)	-554	-3,731	1,414	271	-2,395	-2,613	672
	Theo – MBT	-1,960	-4,514	485	2,675	-2,298	-5,637	-658
5	Exp (IS)	-2,520	-13,800	1,590	3,330	-2,460	-2,880	-840
	Exp (ES)	-2,790	-8,280	1,530	5,760	-2,580	-4,200	-1,020
	Theo – FE (ES)	-1,500	-6,473	1,760	1,753	-5,870	-1,327	-2,008
	Theo – MBT	-4,588	-9,601	1,794	2,641	-2,968	-4,400	-4,595
6	Exp (IS)	-4,380	-7,890	1,620	4,230	-4,770	-8,430	-916
	Exp (ES)	1,140	-5,340	450	5,120	-2,730	-6,720	-270
	Theo – FE (ES)	-720	-4,850	1,838	352	-3,114	-3,397	900
	Theo – MBT	-2,690	-6,066	705	4,158	-3,107	-7,566	-846
7	Exp (IS)	-2,250	-7,380	1,920	4,260	-4,500	-8,880	-922
	Exp (ES)	-1,680	-6,360	690	5,190	-2,790	-8,190	-300
	Theo –FE (ES)	-709	-5,255	1,904	368	-3,227	-4,445	887
	Theo – MBT	-2,690	-6,127	704	3,710	-3,122	-7,700	-878

[a]See Table A-2 for description of load case.

[b]IS = internal spudwell; ES = external spudwell; FE = finite element; MBT = modified beam theory.

[c]1 psi = 6.9 kPa.

[d]See Figure A-9 for location of strain gages.

Table A-4. Statistical Comparisons of Stress Data from Table A-3

Comparison of Data Source[a]	Statistical Derivation	Stress (psi)[b] for Load Case[c] – 1	2	3	4	5	6	7	Totals for All Load Cases (psi)
Exp (ES) to MBT	Median	484	565	610	537	1,321	726	490	565
	Average	528	1,338	662	420	1,523	756	591	831
	Standard Deviation	708	1,666	735	502	1,991	855	751	1,154
Exp (ES) to FE	Median	450	1,195	1,215	672	1,807	1,170	1,187	1,187
	Average	538	1,135	1,647	1,054	2,069	1,711	1,926	1,440
	Standard Deviation	647	1,300	2,042	1,345	2,423	2,329	2,464	1,907
Exp (IS) to MBT	Median	379	772	867	550	1,520	864	1,180	864
	Average	362	1,085	952	817	1,849	1,014	946	1,004
	Standard Deviation	433	1,484	1,040	973	2,363	1,227	1,046	1,342
Exp (IS) to FE	Median	269	1,615	1,364	856	1,553	3,040	1,809	1,615
	Average	372	1,562	1,330	1,116	2,318	2,757	2,156	1,659
	Standard Deviation	511	1,669	1,823	1,407	3,221	3,142	2,580	2,244
FE to MBT	Median	365	1,634	1,889	1,330	2,902	1,216	1,765	1,634
	Average	679	1,842	2,034	1,235	2,243	2,007	1,789	1,690
	Standard Deviation	876	2,173	2,464	1,697	2,526	2,437	2,105	2,111
Exp (IS) to Exp (ES)	Median	340	360	690	570	270	1,710	930	570
	Average	527	647	485	682	1,414	1,749	900	915
	Standard Deviation	705	882	580	774	2,338	1,952	1,037	1,340

[a]ES = external spudwell; IS = internal spudwell; FE = finite element; MBT = modified beam theory.

[b]1 psi = 6.9 kPa.

[c]See Table A-2 for description of load case.

(45,800-kg) crane are shown in Figure A-13. The measured deflections compare favorably with the conventional beam theory data; however, the general shape of the experimental deflection curve suggests the end of the causeway is restrained by the adjacent causeway. This condition is possible since the two causeways are interlocked by end connectors. Most importantly is the fact that the magnitudes are similar, which probably means the beam theory predictions are pretty good.

The critical members in the internal and external spudwells were strain-gaged to determine load capacities. The test results are reported in the spudwell section of this report.

The crane platform was strain-gaged in two locations. One gage measured the strains in the reinforced angle, and the other one measured the strains at a location near the center spudwell. The gage on the reinforced angle received very little strain, because the outrigger was positioned between two piles so that the load went directly into the piles. The gage near the center spudwell measured stresses approaching 5,000 psi (35,000 kPa). The largest stresses occurred when the crane transferred from outriggers to wheels, which indicates most of the outrigger load went directly into the pile.

MODIFIED BEAM THEORY

The stress data compiled from the strain gages did not correlate with the finite element model in some cases. Unusual stress patterns caused by structural discontinuities made it difficult to distinguish between

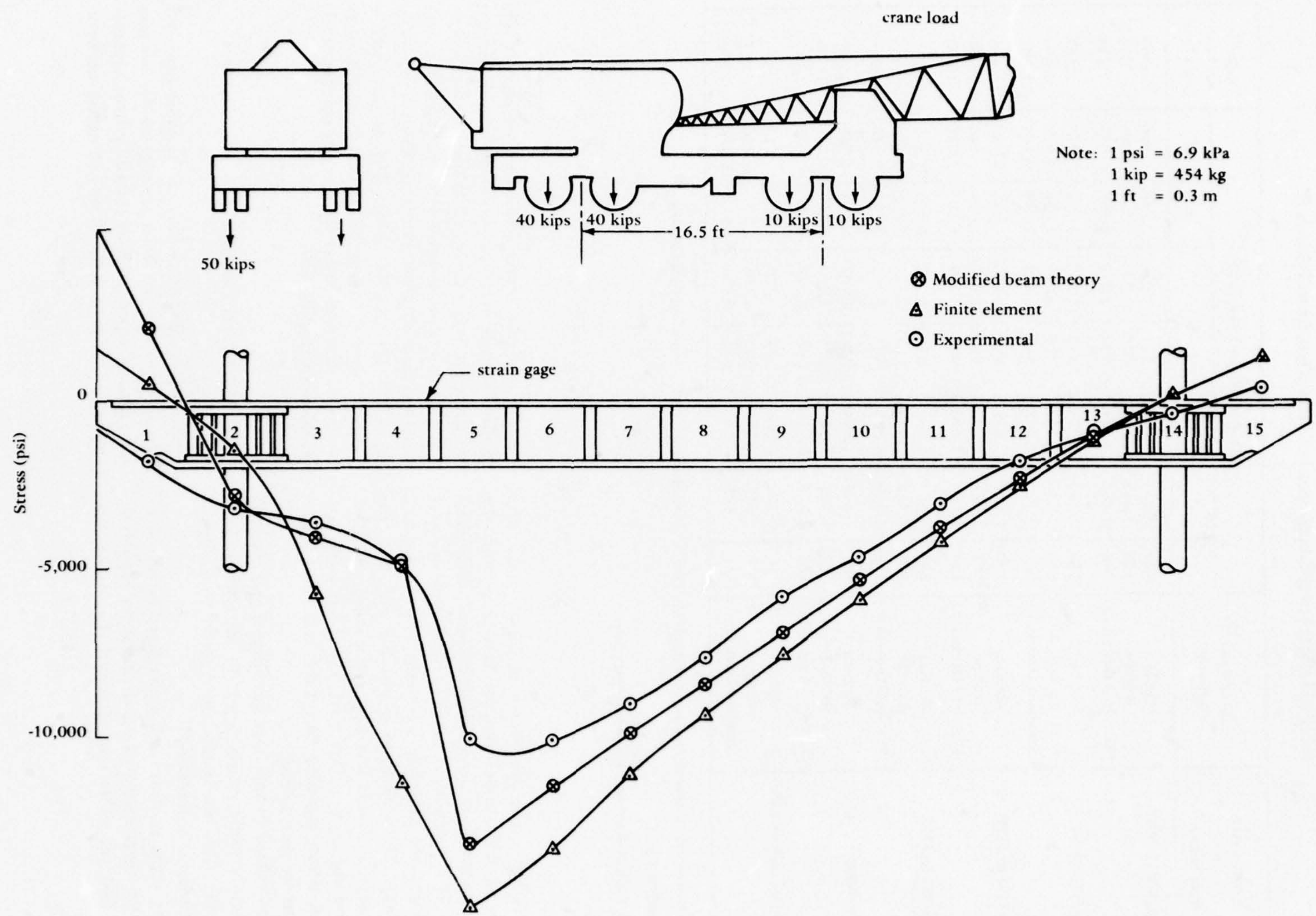


Figure A-10. Stress variation at indicated point as 100,000-pound (45,630 kg) crane drives from end to end of causeway.

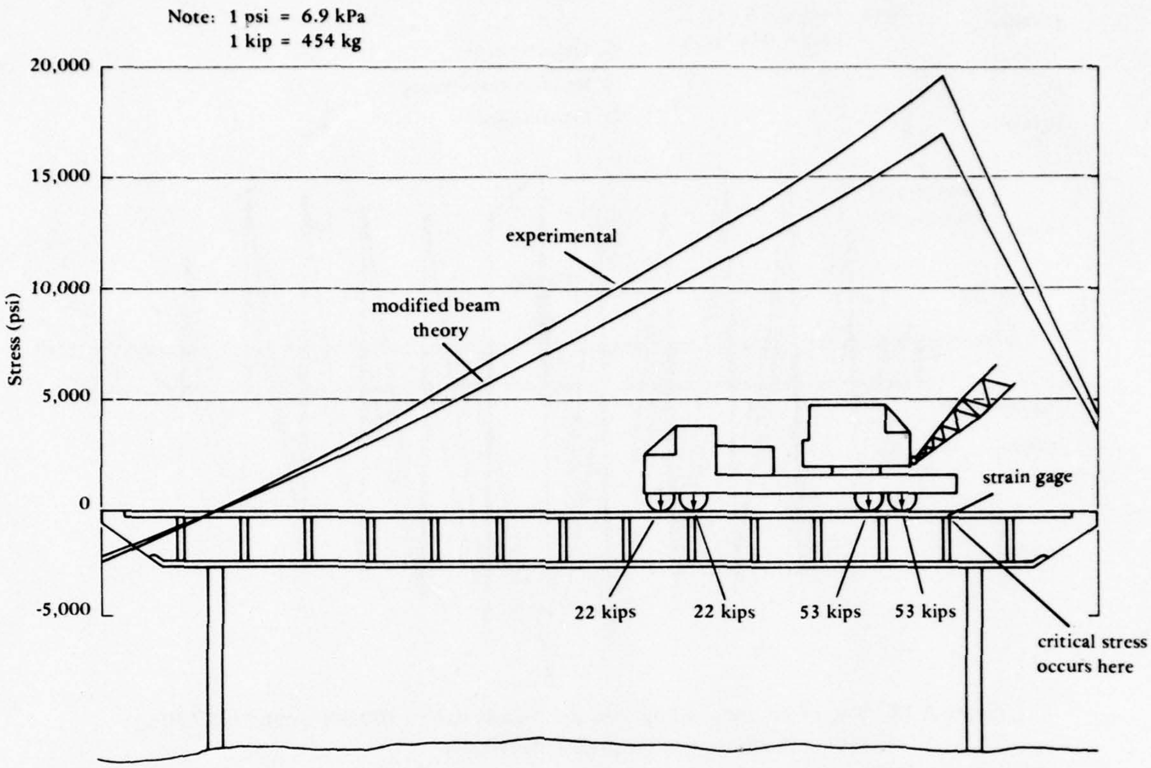


Figure A-11. Stress influence line for P&H 8100 crane on causeway with 6-inch (15.5-cm) angles and four-pile supports.

experimental error and modeling error when comparing like-load stresses. Two apparent reasons for variable stresses are (1) local stresses due to point loads, and (2) the degree of restraint offered by the piles in the spudwells. Consequently, it was decided to re-examine the beam theory as a prediction technique. The finite element model had provided insight into the flow of forces, shears, and moments in the causeway structure. A simplified analysis based on external beam shears and moments was developed to compare experimental and finite element results.

The logical approach was to begin with the basic beam theory where stresses are constant at a given distance from the neutral axis at each cross section. The finite element study showed that external shear, **V**, induces a local bending moment in the angles at the 9-inch (23-cm) openings between pontoons. This local bending moment phenomenon was analyzed. Figure A-14a shows a cross section at the 9-inch (23-cm) opening resisting pure moment, while Figure A-14b shows the same section resisting pure shear. The open section is efficient for resisting moments, but inefficient for shears.

Figure A-15 shows the probable effect of a load, or shear, on a single angle and a model of the connection. An examination of the restraint offered by the clamping action of the bolts and plates led to the "effective" length, ℓ, of 11.75 inches (0.30 m) shown on the model. Therefore, the moment, **m**, induced in the angles at the ends due to a shear load, v, is

$$m = v\ell/2 = 5.875\,v \tag{1}$$

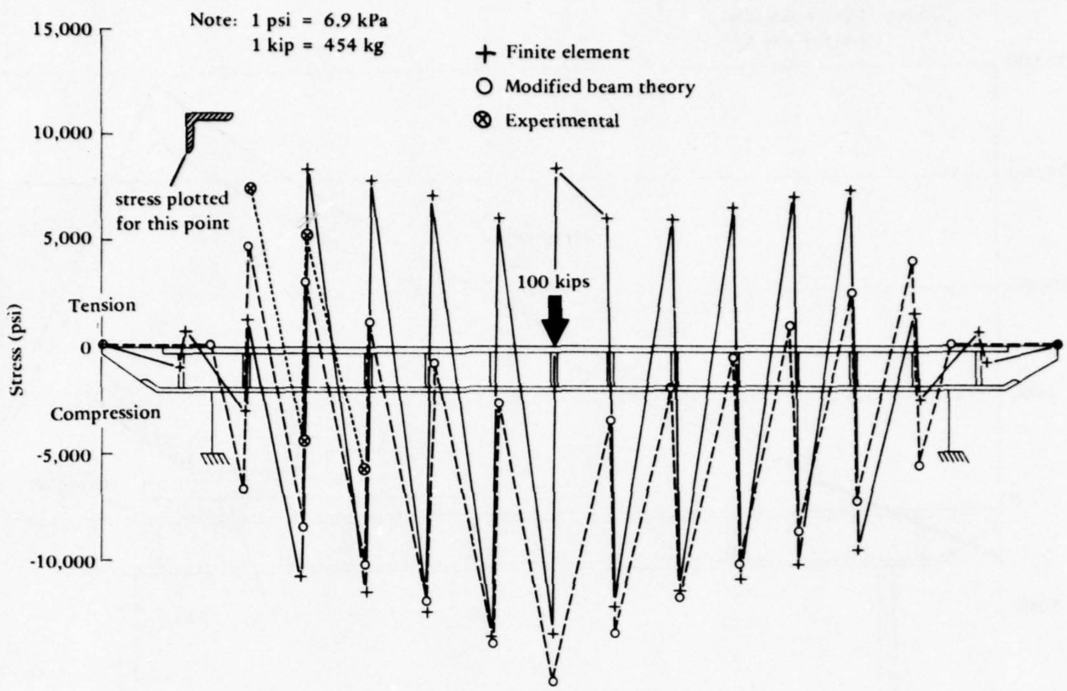


Figure A-12. Stress variations of upper pontoon angle due to 100,000-pound (45,630-kg) load – theoretical and experimental results.

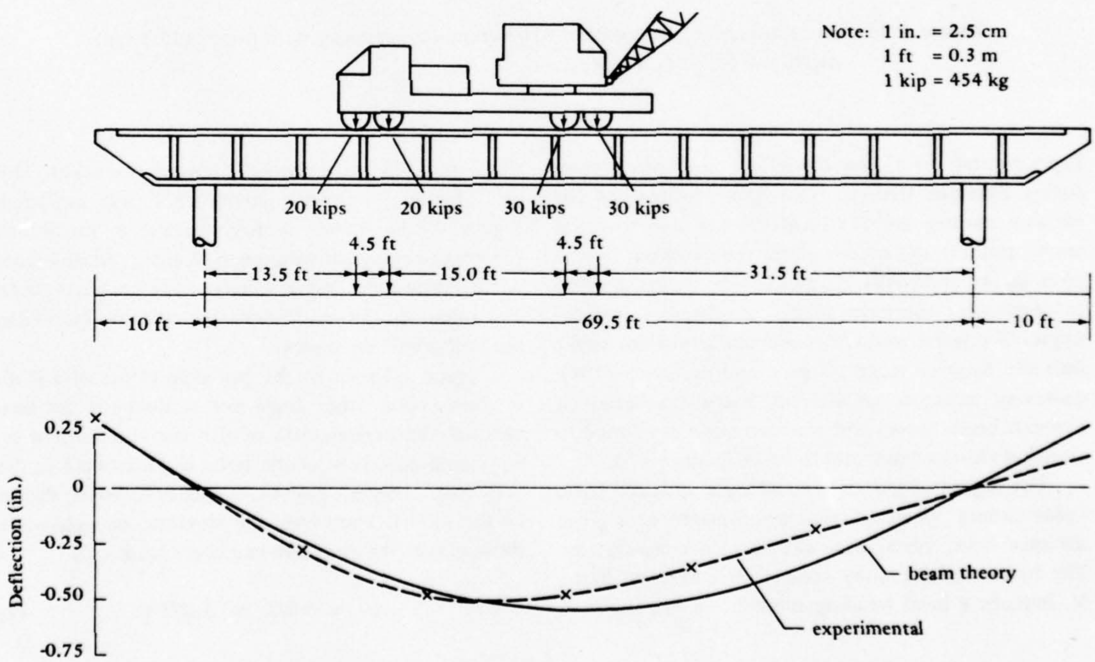
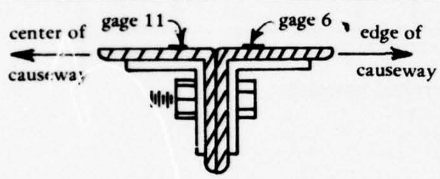


Figure A-13. Deflections of pile-supported 3x15 pontoon causeway with 100,000-pound (45,630-kg) crane.

Table A-5. Stresses Measured on Back-to-Back Structural Angles of Causeway

Load Case	Measured Stresses[a] (psi)			
	Internal Spudwell		External Spudwell	
	Gage 11	Gage 6	Gage 11	Gage 6
Rear bogie of crane on end pontoon	1,830	1,920	1,830	1,290
Rear bogie of crane on fourth pontoon	1,410	2,700	870	1,530
Rear bogie of crane on eighth pontoon	780	1,710	180	420
Outriggers of crane on pontoons 5 and 8	900	1,500	0	390
Outriggers of crane on pontoons 7 and 10	750	1,920	780	690
Dozer on end pontoon	1,260	2,460	1,440	1,470
Dozer on fourth pontoon	570	0	120	120
Dozer on sixth pontoon	450	1,290	390	570
Dozer on eighth pontoon	240	810	140	420

[a]



[b] 1 psi = 6.9 kPa.

Again, assuming that beam theory applies, the shear is divided equally among the 12 angles at a given causeway cross section. The total external shear is the sum of the shears in each angle and is related to the external shear as

$$v = \frac{V}{12} \qquad (2)$$

Combining Equations 1 and 2 gives

$$m = 5.875\left(\frac{V}{12}\right) = 0.489\,V \qquad (3)$$

The elastic stress, f_b, due to bending of the angles is

$$f_b = \frac{m}{S} \qquad (4)$$

where S is the section modulus of a 6 x 6 x 1/2-inch (15.5 x 15.5 x 1.27-cm) angle. Now, the local bending stress, f_b, induced in the angle by shear is given by

$$f_{b_2} = \frac{0.489\,V}{11.845\text{ in.}^3} = \frac{V}{24.223} \quad \text{(minimum stress)}$$

$$f_{b_1} = \frac{0.489\,V}{4.606\text{ in.}^3} = \frac{V}{9.419} \quad \text{(maximum stress)} \qquad (5)$$

where f_b is in psi, and V is in pounds.

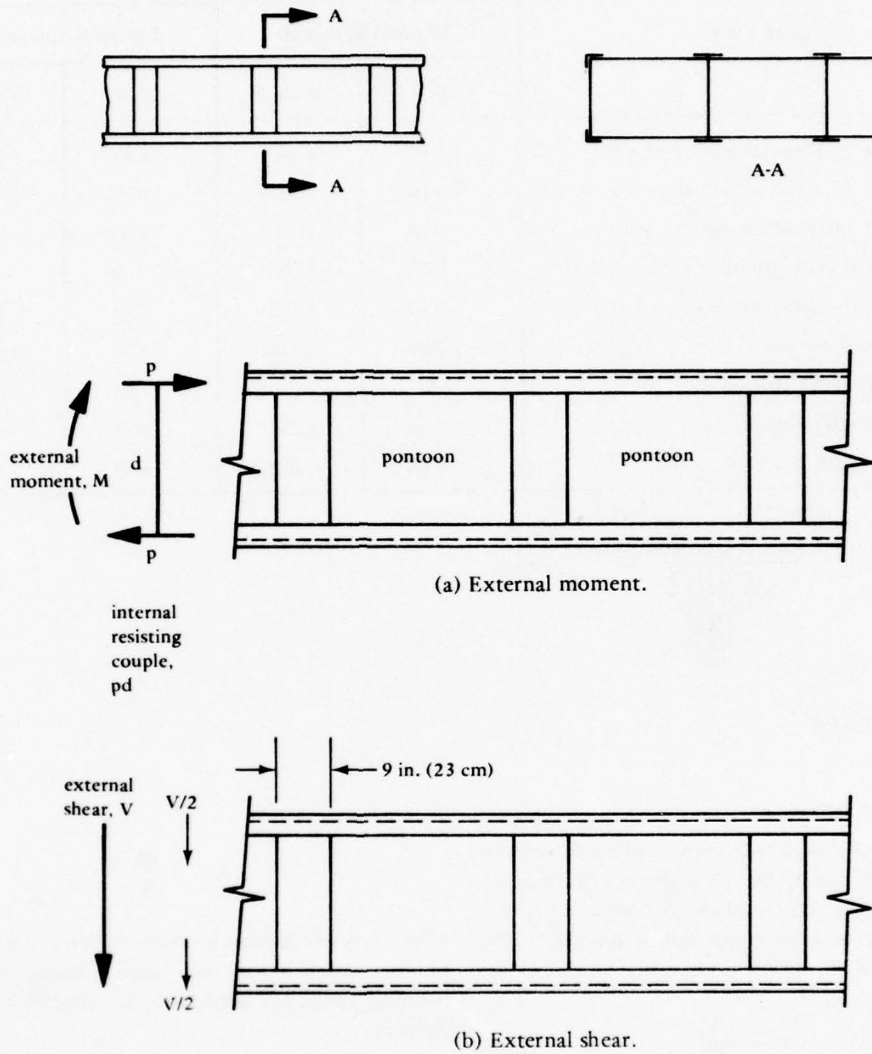
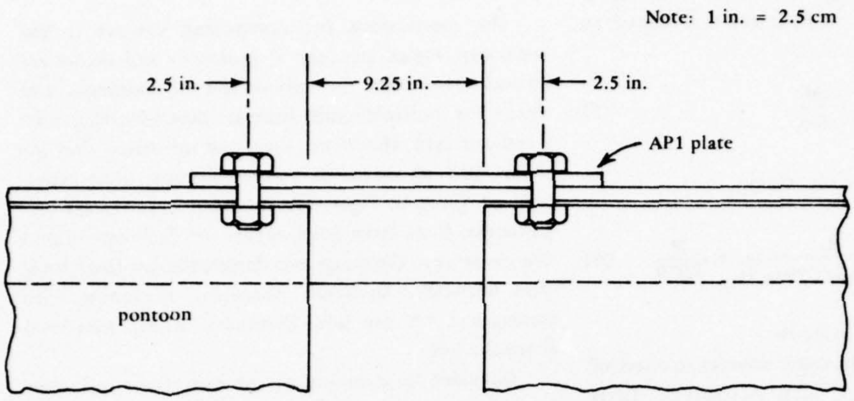
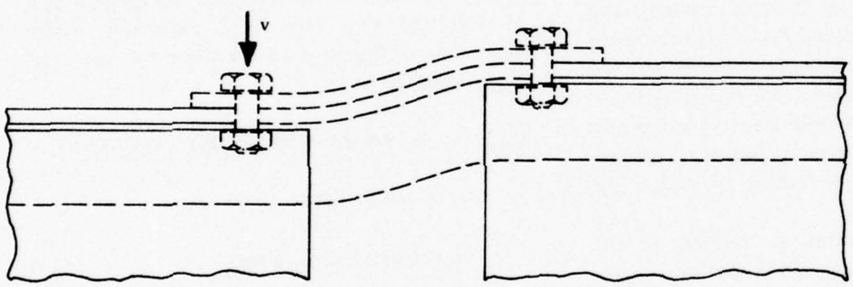


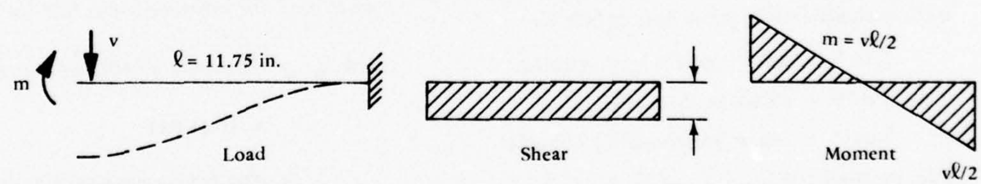
Figure A-14. External moments and shears on cross section of causeway.



(a) Detail of 9-inch opening.



(b) Deflected shape of angle due to shear-type load.



(c) Model of connection at 9-inch opening.

Figure A-15. Model of connection angle at 9-inch (23-cm) opening between pontoons.

The total stress in the angle is the algebraic sum of the shear-induced bending stress and the axial stress, f_a, due to external moment, **M**. As shown in Figure A-14, the external moment is resisted by the couple formed by axial forces in the six pairs of top and bottom causeway angles at any given cross section. The axial load, **p**, in each angle is related to the external moment as:

$$p = \frac{M}{6d} \tag{7}$$

where **d** = 60 inches = angle separation,

and $$f_a = \frac{P}{A} = \frac{M}{(6)\,(60\text{ in.})\,(5.75\text{ in.}^2)} = \frac{M}{2{,}070} \tag{8}$$

where f_a is in psi, and **M** is in in.-lb.

The expressions relating angle stresses to external moments and shears have been developed. There remains the task of combining the axial stress with the local bending stresses. Since the bending stress has a maximum and a minimum value at each point, it is necessary to consider signs of external moments and shears as well as the specific location where the stress is to be computed. It has been determined that the maximum stresses in the angles do indeed occur at the point where the angles bolt into the pontoons. Inspection of the possible stress conditions on each side of the 9-inch (23-cm) opening provided the information for Table A-6.

Positive external moment is defined as the moment that causes compression in the top angles. Positive external shear acts upward on the left side of a free-body section. The location of a point on an angle is designated by subscripts as follows:

L or R = left or right side of opening

T or B = top or bottom angle

1 or 2 = toe or horizontal leg of angle

For example, f_{bLB1} is interpreted as the shear-induced bending stress on the left side of the toe of the bottom angle. Since there are eight different locations, eight expressions are required for combining the axial and bending stresses at any one 9-inch (23-cm) opening.

Frequently, the maximum stress at an opening is desired. In that case, the stress can be taken as the sum of the absolute values of the bending and axial stress, or

$$f_{max} = \left|\frac{M}{2{,}070}\right| + \left|\frac{V}{9.419}\right| \tag{9}$$

The expressions for computing stresses in the causeway angles are easy if moments and shears are known. However, the calculation of moments and shears for multiple loads, such as crane wheels, can be a tedious task; therefore, unit load influence lines for shears and moments in a four-pile-supported causeway are given in Figures A-16 and A-17, respectively. Influence lines have been drawn for each opening in the causeway. Openings are designated by their location between numbered pontoons, beginning with pontoon 1 on the left. Distances, x, are measured from the left.

Consider an example of the use of the influence lines. Suppose you want to know the shear and moment at the opening between pontoons 3 and 4 due to the crane loading shown earlier (40 kips each at x = 20 feet and x = 24.5 feet, and 10 kips each at x = 37 feet and x = 41.5 feet). Referring to the influence line of Figure A-16 for shear 3-4, the total shear is computed as

$$\begin{aligned} V_{3\text{-}4} &= 40(0.850) + 40(0.785) + 10(0.610) \\ &\quad + 10\,(0.545) \\ &= 76.95\text{ kips (34.9 Mg)} \end{aligned} \tag{10}$$

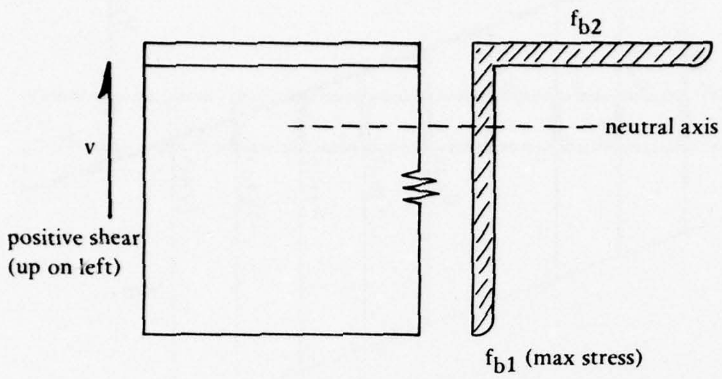
Moments are calculated from the influence line of Figure A-17 for moment 3-4 as follows:

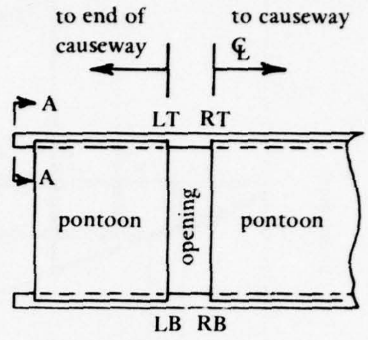
$$\begin{aligned} M_{3\text{-}4} &= 40(6.60) + 40(6.06) + 10(4.59) \\ &\quad + 10\,(4.04) \\ &= 593\text{ ft-kips (804 kN·m)} \end{aligned} \tag{11}$$

The angle stress at any point between pontoons 3 and 4 can now be calculated using the appropriate formula in Table A-6. The maximum stress can be calculated by Equation 9 as:

$$f_{max} = \left|\frac{593\,(12)}{2{,}070}\right| + \left|\frac{76.95}{9.419}\right| = 11{,}600\text{ psi (80,000 kPa)} \tag{12}$$

Table A-6. Expressions for Combining Stresses in 6 x 6 x 0.5-Inch (15.5 x 15.5 x 1.27-cm) Causeway Angles Due to External Moments and Shears





Typical Causeway Elevation

Section A-A

$$f_{LT1} = -[(M/2{,}070) + (V/9.419)]$$

$$f_{LT2} = -(M/2{,}070) + (V/24.223)$$

$$f_{RT1} = -(M/2{,}070) + (V/9.419)$$

$$f_{RT2} = -[(M/2{,}070) + (V/24.223)]$$

$$f_{LB1} = (M/2{,}070) + (V/9.419)$$

$$f_{LB2} = (M/2{,}070) - (V/24.223)$$

$$f_{RB1} = (M/2{,}070) - (V/9.419)$$

$$f_{RB2} = (M/2{,}070) + (V/24.223)$$

and the dead load stress is given in Figure A-5 as 5,600 psi (38,600 kPa) for a total stress of 17,200 psi (119,000 kPa). This figure compares to 18,700 psi (129,000 kPa) for the finite element solution.

Dead load stresses were not measured experimentally, but the crane live load stresses between pontoons 3 and 4 for the three methods are:

Finite element	11,200 psi (77,200 kPa)
Modified beam theory . .	11,600 psi (80,000 kPa)
Experimental	8,300 psi (57,200 kPa)

In short, the modified beam theory provides a convenient tool for computing stresses in the causeway angles. The influence lines of Figures A-16 and A-17 provide external moments and shears for multiple loads. With the moments and shears, maximum stresses can be calculated for that point with Equation 9. Beam theory assumes that the stresses in all the angles at equivalent points of a cross section are equal. Although this assumption may be erroneous in the elastic stress range, the extreme case of total failure probably justifies such an assumption. The major benefit is a relatively simple tool that can be used to estimate critical stresses.

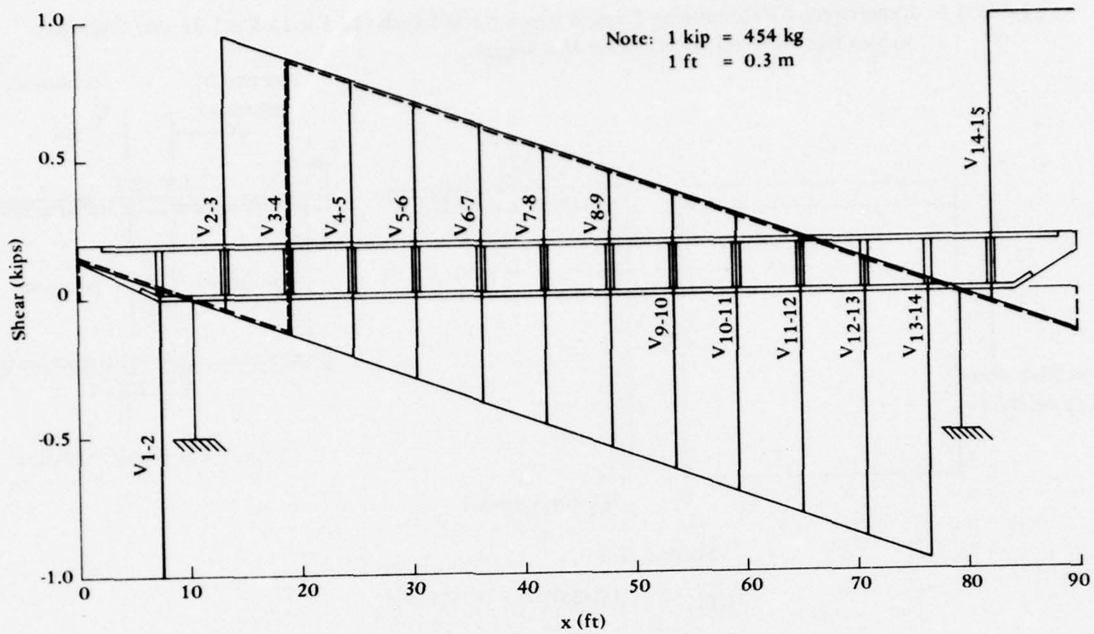


Figure A-16. Influence lines for shear at openings between pontoons.

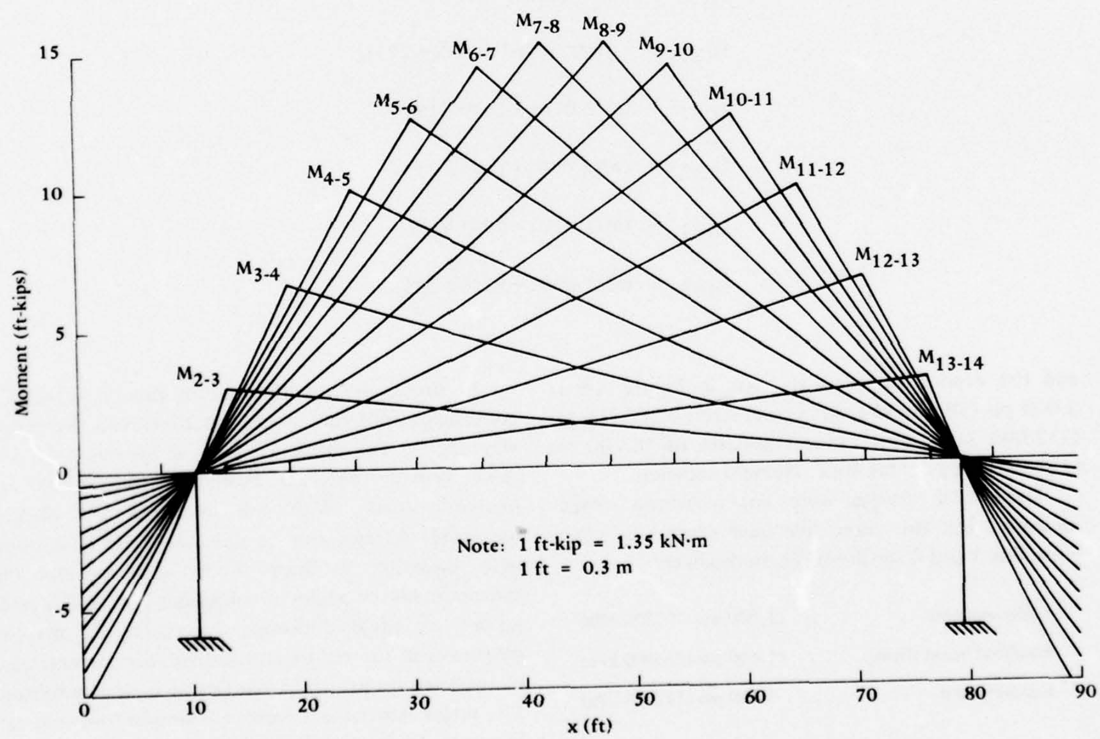


Figure A-17. Influence lines for moment at openings between pontoons.

SAFETY FACTORS AND FAILURE CRITERIA

To determine safety factors for the theoretical analysis, it is necessary to establish failure criteria for the causeway structure. The stresses displayed on the influence diagrams are the maximum stresses that act at the particular cross section. There is a wide range of stress levels at the 9-inch (23-cm) openings of the causeway. The fact that one angle becomes stressed beyond yield means that it will pass some of its load to its neighbor and so on until all angles at a cross section have yielded to form a plastic hinge mechanism. Consequently, the true failure load does not occur at the time the angle first yields. This leads us to the point where one must define what is really meant by failure.

True failure means collapse of the structure. Figure A-18 shows a collapsed mechanism. At this stage the angles at the collapsed section have totally yielded. Figure A-19 shows an angle subjected to pure bending. When one compares the moment an angle can resist at first yield to the load it can carry with a fully yielded cross section, it is shown that the angle has a reserve capacity of m_p/m_y = 268 in.-kips/166 in.-kips = 1.61 or about 60 percent over and above the yield moment.

One now computes the factor of safety of an angle in pure bending subjected to an extreme fiber stress of 20,000 psi (138,000 kPa). The moment capacity of the angle at f = 20,000 psi is

$$M_{20,000} = fS = 20{,}000 \text{ psi} \times 4.61 \text{ in.}^3 = 92.2 \text{ in.-kips} (10.4 \text{ kN·m}) \quad (13)$$

Now, the ultimate factor of safety is given by (ultimate moment)/(applied moment) or

$$\text{Factor of Safety} = \frac{268 \text{ in.-kips}}{92.2 \text{ in.-kips}} = 2.91 \quad (14)$$

The causeway angles are subjected to the type of bending described above by transverse vertical shear loads. In addition, primary bending moments applied to the causeway induce axial loads in the structural angles. Therefore, the causeway angles are subjected to combined axial load, shear, and bending, as depicted by the free-body diagram of an angle element in Figure A-20. In Figure A-20, p is the axial load, v is the transverse load (shear), and m is the moment. These combined forces, which are in equilibrium, cause failure of the angle. How does failure occur? Obviously, an axial load stress (p/A) greater than yield will fail the angle. Calculations indicate that shear stress, as computed by classical methods, is less critical than the shear-induced moment stresses; therefore, angle failure will result from combined axial and bending stresses. As shown previously, pure bending failure occurs when the section is fully yielded. The combined effect of axial load and bending interaction is shown in Figure A-21 for a 6 x 6 x 1/2-inch (15.5 x 15.5 x 1.27-cm) structural angle. Note that the moment and axial loads are translated to stress by applying the properties of the angle. Moment stresses greater than yield are fictitious, but indicate the elastic stress level that would be required to develop the plastic moment. Examination of the interaction diagram indicates that the plastic moment (true failure) exceeds the first yield moment (frequently throught of as failure) by 60 percent. For subsequent references to failure of the causeway structure see the plastic criterion in Figure A-21. The exact safety factor will vary from 1.6 to 2.9 according to the relative magnitude of applied moments and shears.

The criteria established for a single angle can be extrapolated to the entire causeway structure. Figure A-22 shows the results of such an extrapolation, where the interaction diagram is expressed in terms of the external moments and shears acting on the causeway. Consider an example where the external moment is 2,000 ft-kips (2.7 MN·m). Using the failure criterion described above, the shear could be as high as 370 kips (167,800 kg), compared to 190 kips (86,200 kg) for the yield criterion, and 65 kips (29,500 kg) for the allowable criterion. Since combined bending and axial loads cause failure before shear, the failure criterion should be based on bending plus axial stresses as described above.

STRUCTURAL CONSIDERATIONS FOR A CRANE PLATFORM

Usually a crane is designed so that each dual wheel supports about 20,000 to 25,000 pounds (9,100 to 11,300 kg). There are some cases, such as

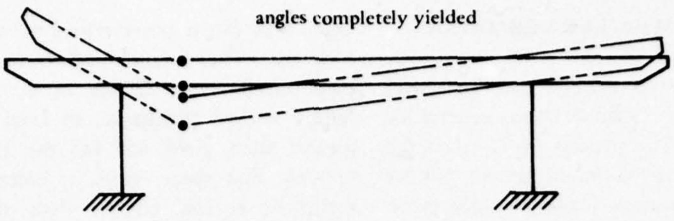
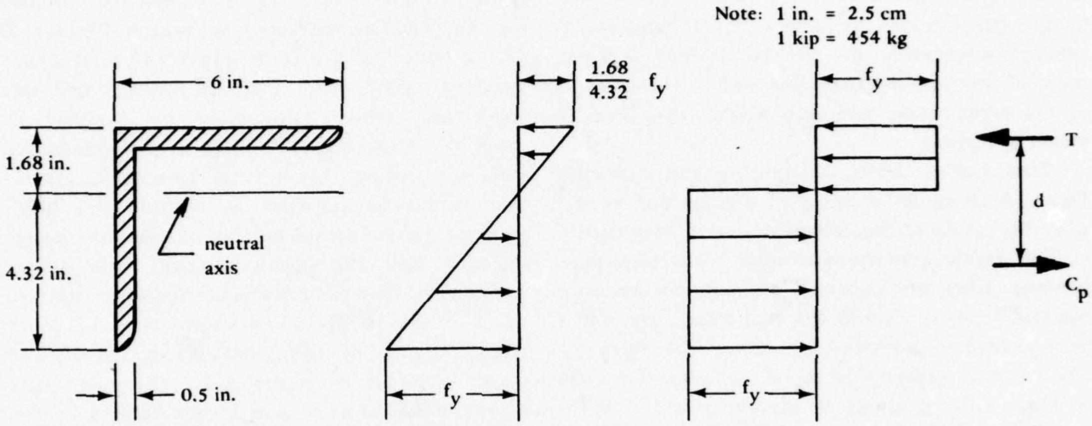


Figure A-18. Collapse mechanism for an elevated causeway.



Angle Cross Section

<u>First Yield</u>

$m_y = f_y S_x$

$= 36.40(4.61)$

$= 166$ in.-kips

<u>Fully Yielded</u>

$m_p = C_p d = (f_y)(4.32 \times 0.5)(d)$

$= (77.76 \text{ kips})(3.45 \text{ in.})$

$= 268$ in.-kips

Figure A-19. Elastic (first yield) and fully yielded stress blocks for a structural angle.

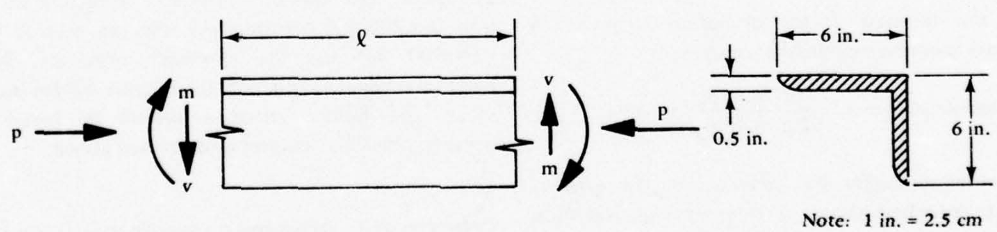


Figure A-20. Free-body diagram of a section of causeway angle between adjacent pontoons.

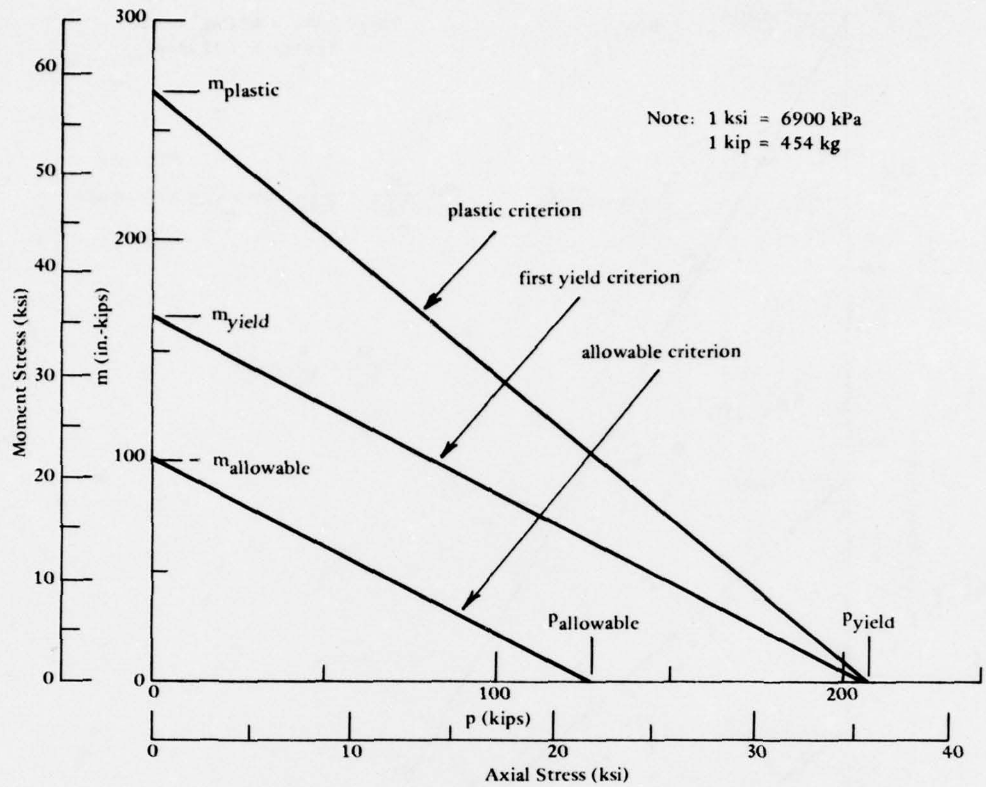


Figure A-21. Axial load and bending moment interaction for a 6 x 6 x 1/2-inch (15.5 x 15.5 x 1.27-cm) structural angle.

the P&H 6250-TC, where each rear dual supports more than 40,000 pounds (18,100 kg). When a crane is handling loads, it operates with outrigger supports to provide greater stability from overturning. The overturning moments of a container load [40,000 pounds (18,100 kg)] at 50 feet (15.2 m) coupled with the weight of the crane [150,000 pounds (68,000 kg)] can develop reactions greater than 100,000 pounds (45,400 kg) on one outrigger. These loads must be transmitted through the causeway structure into the supporting piles.

Figure A-23 indicates different ways that applied loads can limit the causeway capacity. The first problem is to spread the outrigger load so that it can be taken into the pontoons. During the Phase II tests a built-up float of 12 x 12-inch (30.5 x 30.5-cm) timbers, shown in Figure A-24, was used. The floats were designed to transfer loads up to 150,000 pounds (68,000 kg) into the vertical walls of the pontoons. A load introduced into a pontoon must then be transferred into the structural angle system as depicted in Figure A-23. This capacity, estimated at about 180,000 pounds (81,700 kg), applies to both internal and external spudwells. Finally, moments and shears generated by primary loadings, such as shown in Figure A-23, must be resisted by the structure.

It was apparent that the crane platform would have to be supported by more than four piles. The crane outrigger reactions plus the causeway weight could exceed the spudwell design load. A six-pile crane platform was planned; however, the combination of outrigger reactions and causeway deadweight approaches the design capacity of the center spudwells. The external spudwells that retain the

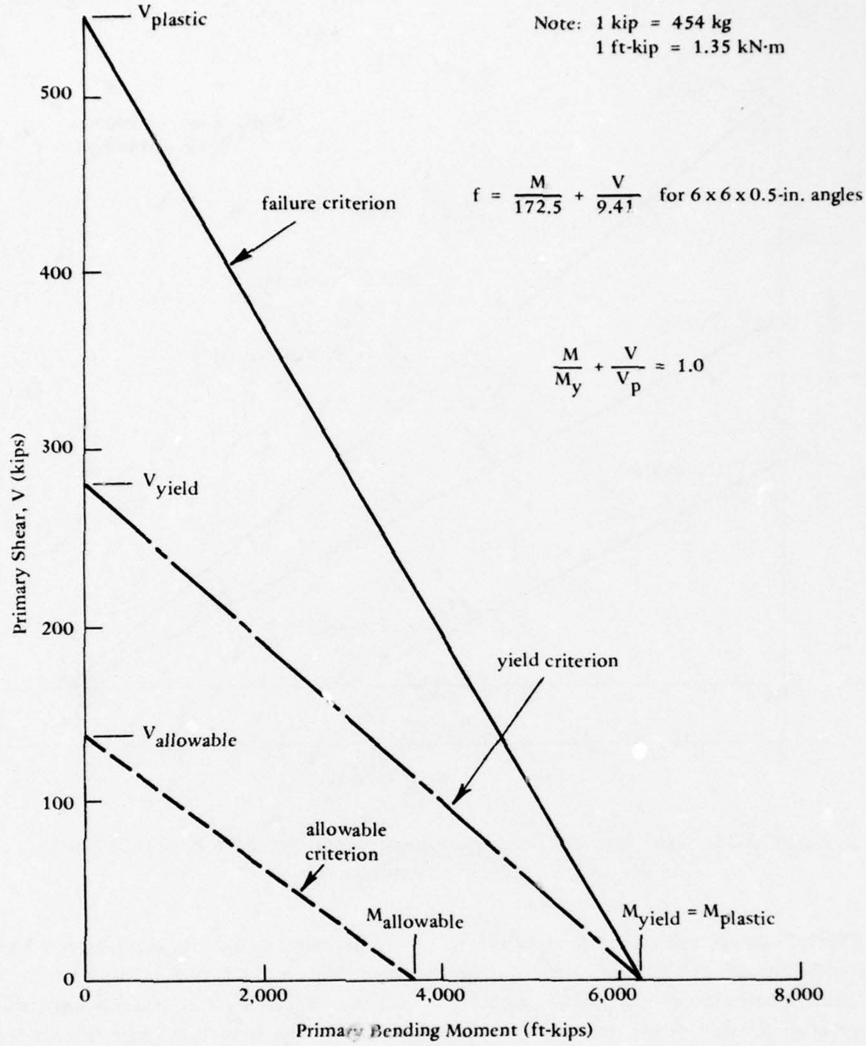
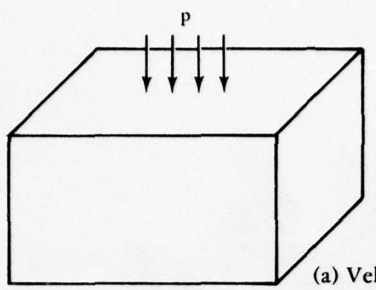


Figure A-22. Interaction diagram for external bending moment and shear loads on causeway structure with 6-inch (15.5-cm) angles.

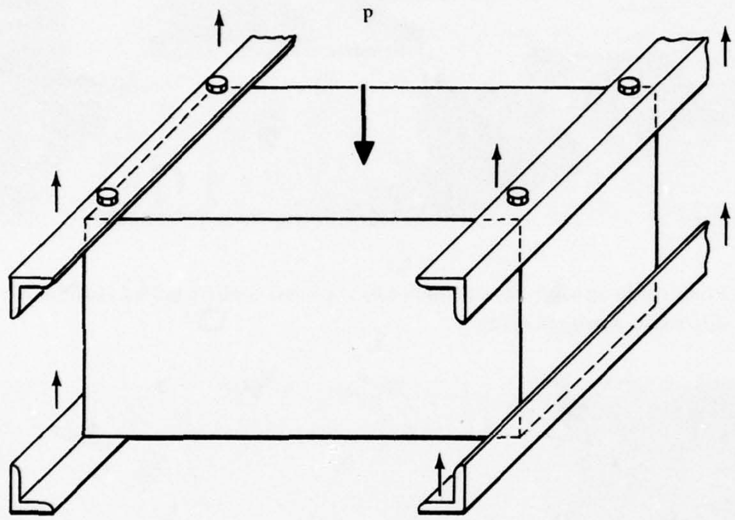
fender piles are not highly loaded most of the time, so they could be used as a secondary support to double the load capacity at the outside edge of the platform. This setup was used in Coronado; the platform support arrangement can be seen in Figure 12. Using fender piles as the only support at the edge of the platform could result in collapse of the platform if accidental mooring loads failed the fender piles. However, if they were used only as secondary supports, their failure would not lead to collapse of the causeway structure.

Additional support members for the crane platform do not reduce the shear requirements of the structural angles. The angles must be capable of transferring loads to the extra spudwells, or the spudwells are of no value. To increase the shear capacity of the crane platform, the exterior angles of the crane platform were reinforced, as shown in Figure A-25, for the Phase II tests. The addition of this plate increases the capacity of the angles to transfer shear across the 9-inch (23-cm) opening by more than four times; however, it does not increase the capacity to transfer loads from pontoons to angles.

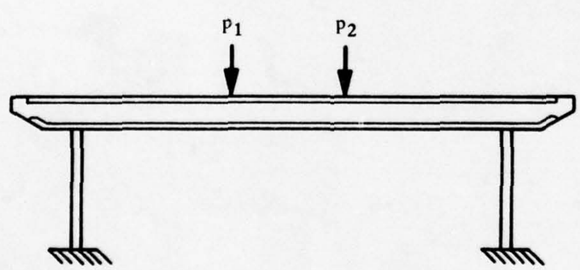


(a) Vehicles with high wheel loads or crane outriggers may cause local deck failure.

(either deck plating between internal stiffeners or the internal stiffeners)



(b) Spudwell loads may exceed the capacity to transfer loads to the structural angle system.



Primary Loads

(c) Heavy vehicle loads may exceed the capacity of the structure to resist external moments and shears.

Figure A-23. Types of loads that limit the load capacity of the elevated causeway.

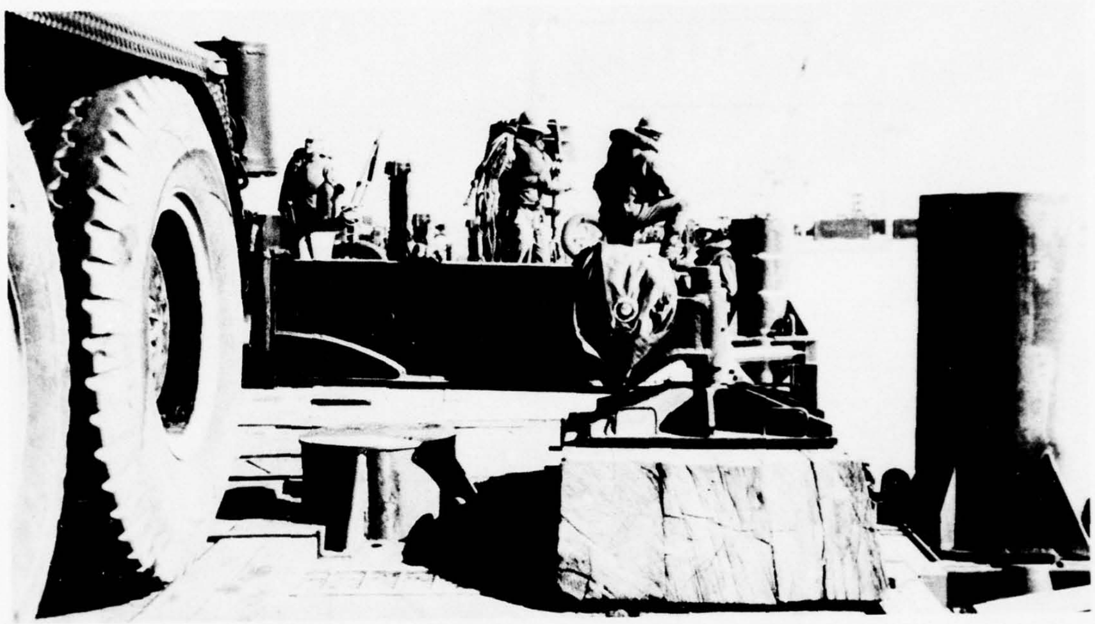


Figure A-24. Float made up with 12 x 12-inch (30 x 30-cm) timbers bolted together to distribute outrigger loads.

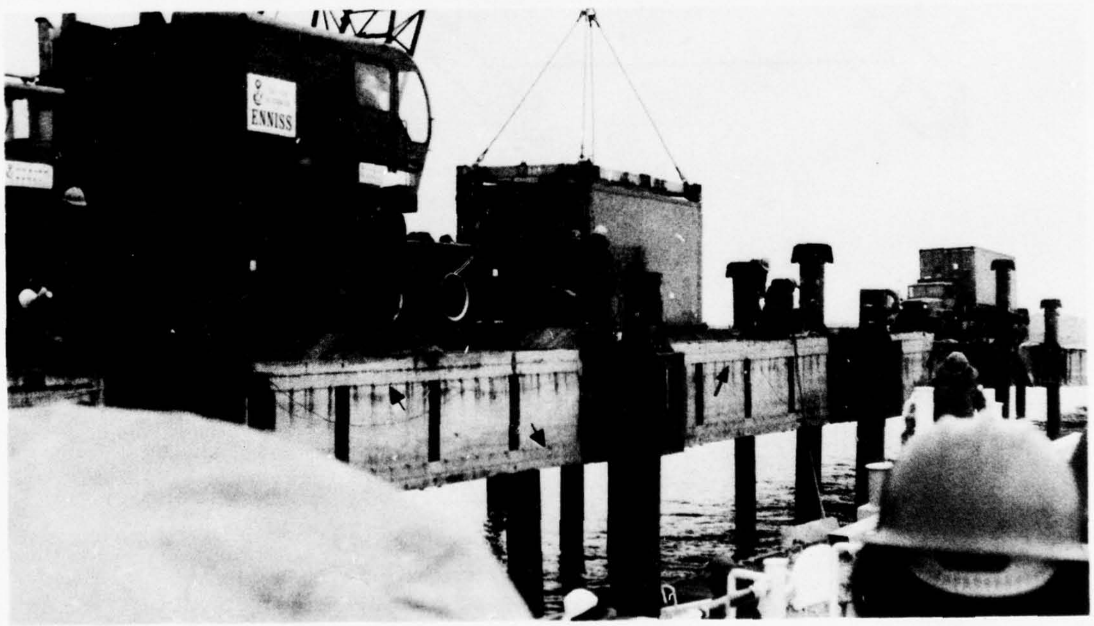


Figure A-25. Reinforcement of causeway angle for crane platform.

The capacity of the causeway to resist primary loads from vehicles is limited by the structural angles. The standard causeway with 6 x 6 x 1/2-inch (15.5 x 15.5 x 1.27-cm) angles can handle 207,000 pounds (93,895 kg) pure shear and 3,795,000 foot-pounds (5.1 MN·m) pure moment. Pure shear or pure moment capacity is reduced by interaction of the two. The primary load capacity of the elevated causeway can be increased by using 8 x 8 x 1/2-inch (20.3 x 20.3 x 1.27-cm) angles in place of the standard 6-inch (15.5-cm) size. The moment capacity is increased by 35 percent, and the shear capacity by 83 percent.

It is possible to exercise some degree of control over the external moment and shears by proper selection of support locations, i.e., where the spudwells will be located. For all practical purposes, it is satisfactory to consider the piling as simple supports that offer little restraint to the causeway because the causeway is much stiffer than the piles. Different support configurations are shown in Figure A-26. For comparison purposes assume a total load, p, which is distributed 60 percent to the rear axle and 40 percent to the front axle, of a crane whose axles are 20 feet (6 m) on centers. Note that the addition of a support at the center of the 70-foot (21-m) span reduces moments by a factor of six, but does not increase the shear. Another option is to load the crane on the short span as in Figure A-26c. In this case the moment is reduced by a factor of four, but the shear increases by about 30 percent. Still another option – and possibly the best way to go – is to align the spudwells directly beneath the crane outriggers. By locating the spudwells under the outriggers, the problem is reduced to transferring the outrigger reactions from the spudwell to the pile, moments and shears disappear as indicated by Figure A-26d. Practically, the spudwells cannot be exactly matched to crane outrigger spacing, but they can be close enough to make it a relatively simple design to transfer the reactions. A second drawback is that the crane would be limited to that specific location. More spudwells could be installed to reduce the load per spudwell.

If a variable crane position on the platform is desired, reinforcements around the spudwell are required to upgrade the shear transfer capability. The rated capacity of each spudwell is 150,000 pounds (68,000 kg). The structure must be capable of transferring the loads from the spudwell. To accomplish this, minimum structural reinforcements similar to those shown in Figure 14 should be performed. These reinforcements become even more critical with 8 x 8 x 1/2-inch (20.3 x 20.3 x 1.27-cm) structural angles so that the full capacity of the angles can be developed. It has been noted that there are some structural inefficiencies in the pontoon/angle system; however, the structure should be able to redistribute loads. The causeway angles in tension are likely to be the first failure point. These reinforcements will not increase the basic strength of the causeway structure, but will effect a more efficient spudwell.

In summary, the outrigger reactions can be distributed to the pontoon deck by timber floats, similar to those used during Phase II tests. Steel beams can be used if clearances are critical. The major limitation of the crane platform is the ability of the spudwell to transfer very large pile reactions to the causeway structure. It was shown that moments and shears can be controlled to some degree by judicious placement of spudwells and crane loads. Shears, which are the least controllable, remain critical. Three approaches to solving the pile load transfer problem were suggested – spudwells can be located at the same spacing as the crane outriggers, reinforcements around the spudwells can be implemented, or more spudwells can be added to reduce the load each spudwell receives.

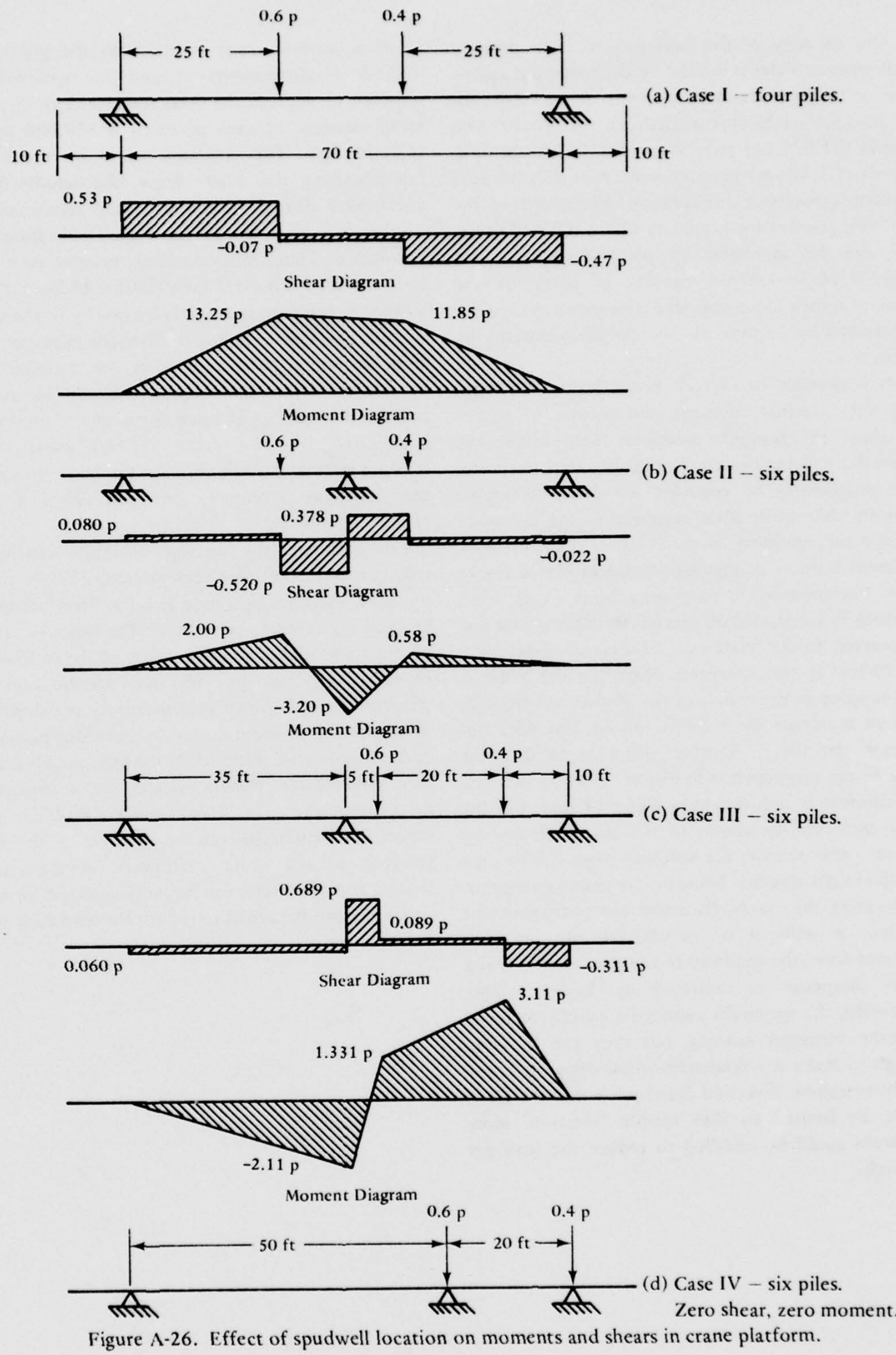


Figure A-26. Effect of spudwell location on moments and shears in crane platform.

Appendix B

DRAWINGS FOR SPUDWELLS AND PILE CONNECTORS

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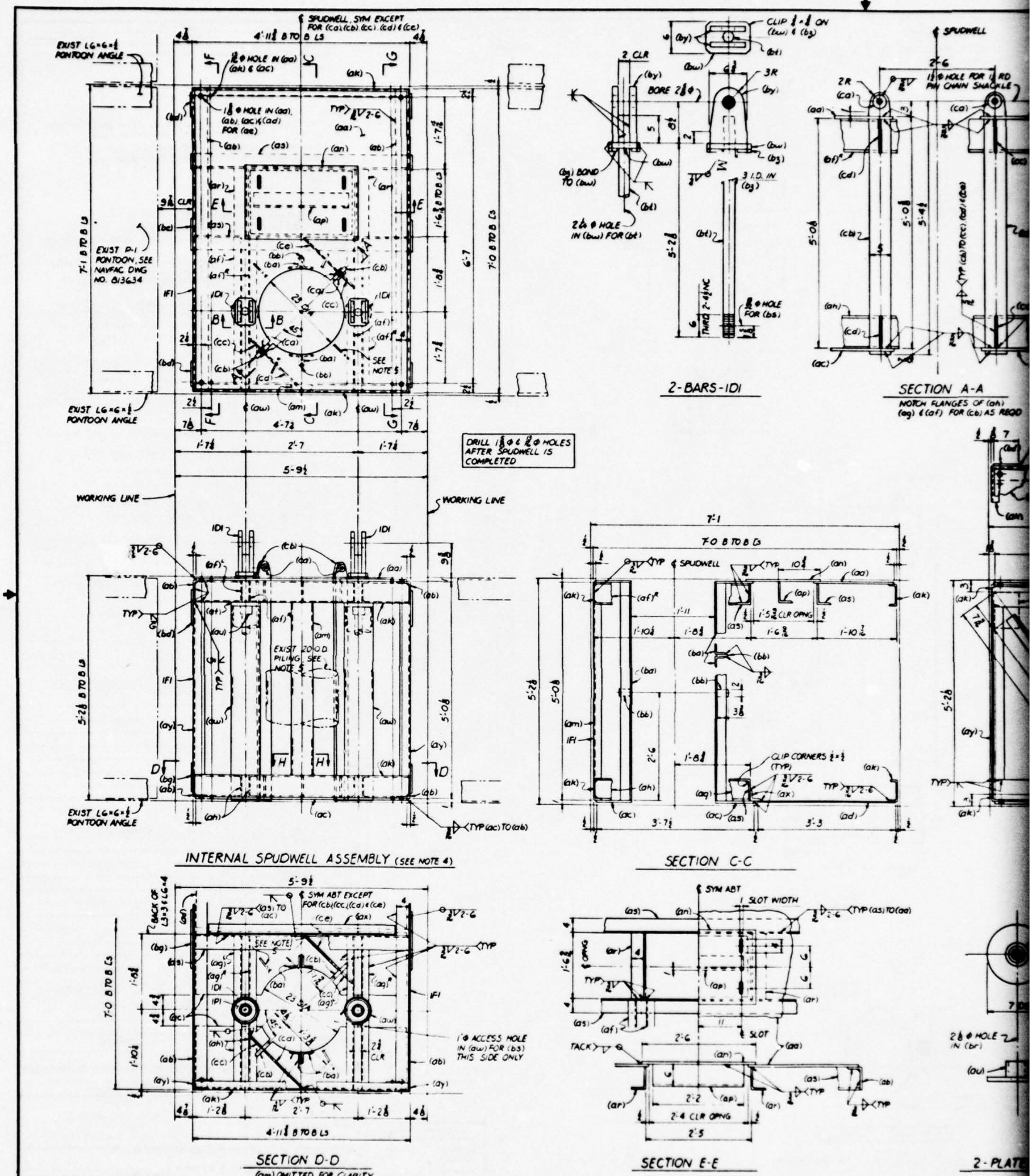
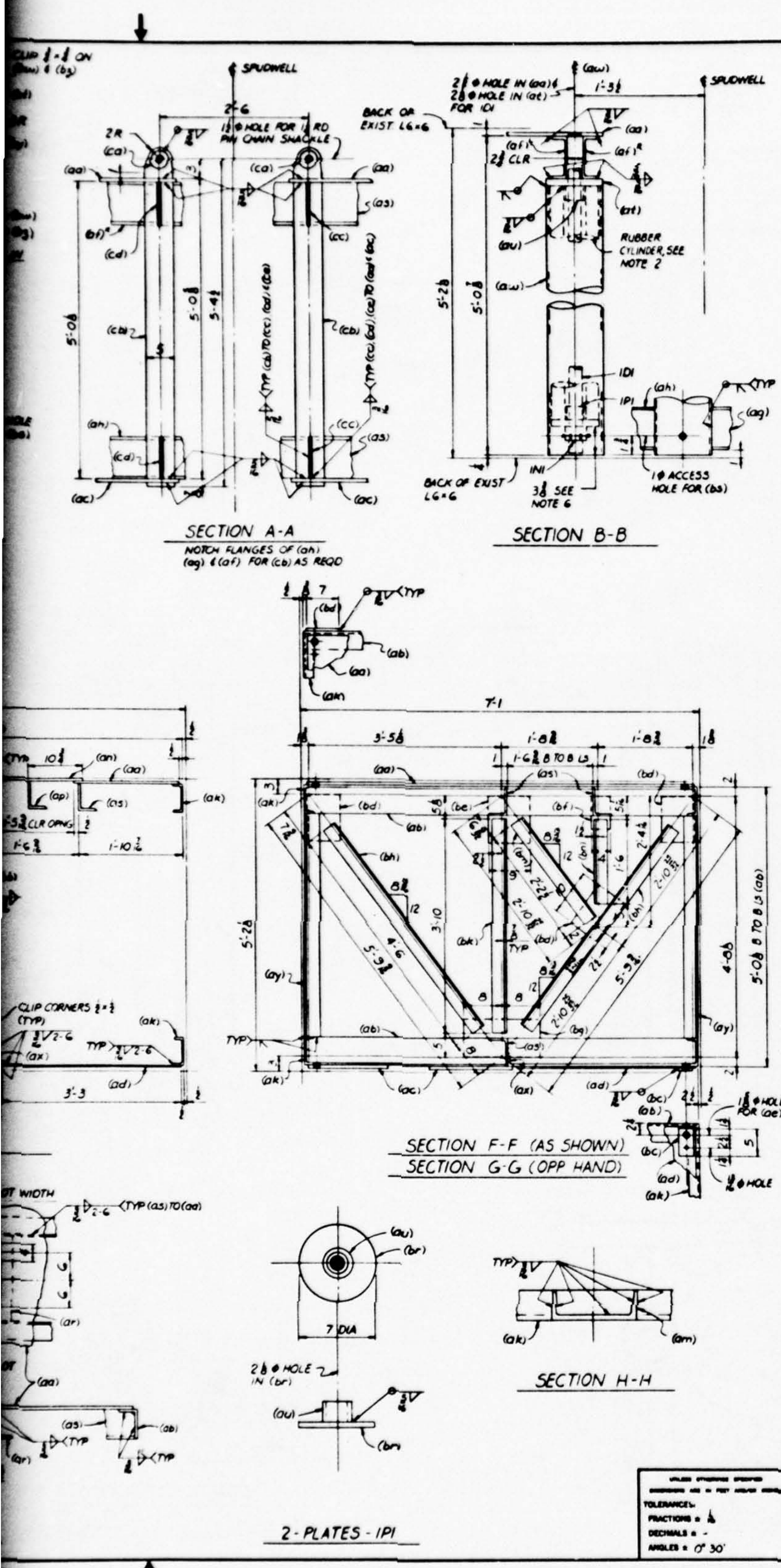


Figure B-1. Internal spudwell.



NOTES

1. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM SPECIFICATION A36 OF MOST RECENT ISSUE UNLESS OTHERWISE NOTED.
2. THE AMPHIBIOUS & HARBOR DIV. CODE L55, WILL FURNISH THE RUBBER CYLINDER WHICH WILL HAVE THE FOLLOWING CHARACTERISTICS:
   - SIZE - 7" O.D. × 3" I.D.
   - LENGTH - 50"
   - DEFLECTION @ 50000 LBS - 5"
   - SPECIFICATIONS - ASTM-D2000 (LINE CALL-OUT) - 3BA720B13C12D11F19
3. RUBBER SHALL CONFORM TO MIL-STD-417, TYPE S, CLASS SB, AND GRADE 705 FOR (bz).
4. CODE L55 WILL DETERMINE PLACEMENT OF INTERNAL SPUDWELL IN EXISTING 3×15 OR 4×15 CAUSEWAY. (REFER TO "PONTOON GEAR HANDBOOK N.L EQUIPMENT P-SERIES" NAVFAC P-401.)
5. OPERATING PERSONNEL SHALL WELD WEDGE-SHAPED PLATES UNDER THE DIRECTION OF CODE L55 FOR MAKING THE FINAL CONNECTION OF SPUDWELL PLATES (aa) & (ac) TO THE 20 O.D. PILING. SEE DETAIL A CEL DWG 74-35-IF.
6. COMPRESS THE RUBBER CYLINDER APPROXIMATELY 1/2" THEREBY PRELOADING IT TO ABOUT 5000 #.
7. SEE CEL DWG 74-35-IF FOR GENERAL PONTOON CAUSEWAY ASSEMBLY.

MARK	NO. REQ'D	SIZE	LENGTH FT	LENGTH IN	REMARKS
	2	HEXAGON SLOTTED NUT, 2"-4 1/2 NC SEMI-FINISHED FORGED, ASTM A325			
		2-NUTS-INI (NOT DETAILED)			
br	2	BAR 7ϕ	0	1	
au	2	TUBE 2 3/8 OD	0	2	2 1/4 ID. AISI-MT-1015 (COLD DRAWN)
		2-PLATES-IPI			
bz	2	CLOSED CELL RUBBER 6 × 8 × 1/4 THICK			SEE NOTE 3
by	4	BAR 6 1/2 × 1	0	11 1/2	
bw	2	BAR 6 × 1	0	8	
bt	2	BAR 2ϕ	5	7 1/8	AISI 1018, THRD ONE END
bs	2	EXTENDED PRONG COTTER PIN 3/8 × 4" LG, STEEL			
		2-BARS-IDI			
ce	2	BAR 6 × 1/2	1	1 1/4	CUT TO FIT (as)
cd	2	BAR 6 × 1/2	1	3 1/8	CUT TO FIT (ak)
cc	4	BAR 3 5/8 × 1/2	0	6	CUT TO FIT (ah) (af) OR (ag)
cb	2	BAR 5 × 1	5	6 1/2	
ca	4	BAR 3ϕ	0	0 3/8	
bn	2	L3 × 3 × 3/8	1	6	
bm	2	L3 × 3 × 3/8	2	2 1/2	
bk	2	L3 × 3 × 3/8	3	10	
bh	4	L3 × 3 × 3/8	4	6	
bg	2	BAR 8 × 3/8	1	4	
bf	2	BAR 4 × 3/8	0	7	
be	2	R 3/8 × 9	0	10	
bd	6	BAR 7 × 3/8	0	10	
bc	2	BAR 3 × 7/16	0	5	
bb	2	BAR 2 × 3/8	0	3	
ba	4	L3 × 3 × 3/8	5	0 3/8	
ay	4	L3 × 3 × 3/8	4	0 3/8	
ax	ONE	L3 × 3 × 3/8	4	3	
aw	2	TUBE 8 OD	4	5 3/8	7 1/4 ID AISI-MT-1020 (D.O.M.)
au	2	TUBE 2 3/8 OD	0	2	2 1/4 ID, AISI-MT-1015 (COLD DRAWN)
at	2	BAR 8ϕ	0	1	
as	3	L6 × 4 × 3/8	4	11	
ar	2	L6 × 4 × 3/8	1	6 3/4	
ap	ONE	L6 × 4 × 3/8	2	2	
an	ONE	R 1/2 × 19 1/2	2	6	
am	ONE	C6 × 13	4	0 3/8	
ak	4	C6 × 13	4	11	
ah	4	C6 × 13	1	7 3/8	
ag	4 2R 2L	C6 × 13	1	6 3/4	
af	4 2R 2L	C6 × 13	3	6 3/4	
ae	8	BOLT 1 1/2 ϕ	0	3 1/2	ASTM A325, HEX HD W/NUT
ad	ONE	R 3/8 × 39	4	10 3/4	
ac	2	R 1/2 × 17	4	10 3/4	
ab	4	L6 × 4 × 3/8	7	0	
aa	ONE	R 1/2 × 58 3/4	6	11	
		ONE-FRAME-IFI			

BILL OF MATERIAL

UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN FEET AND/OR INCHES
TOLERANCES:
FRACTIONS ± 1/16
DECIMALS ± -
ANGLES ± 0° 30'

PROJ NO. YF 53 536 108 01.011
CEL DWG NO. 74-27-IF
DES K MACK | DR K MACK

DEPARTMENT OF THE NAVY — NAVAL FACILITIES ENGINEERING COMMAND
CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA

NL EQUIPMENT
INTERNAL SPUDWELL
FOR
NL PONTOON CAUSEWAYS

SIZE F | CODE IDENT NO 80091 | SCALE NONE | SHEET 1 OF 1

J74-11, J75-19

REV. 1

Figure B-1. Internal spudwell.

2

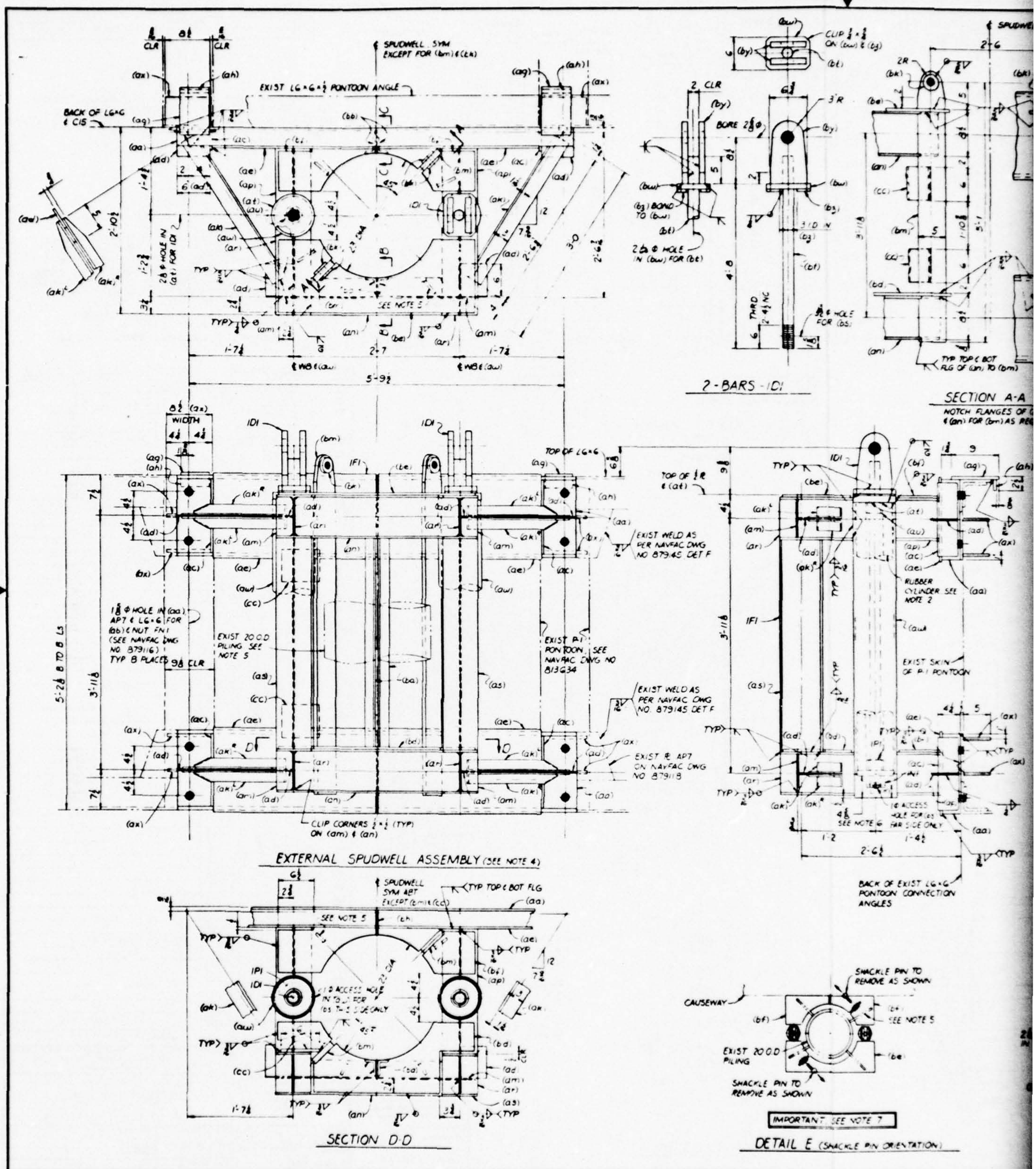
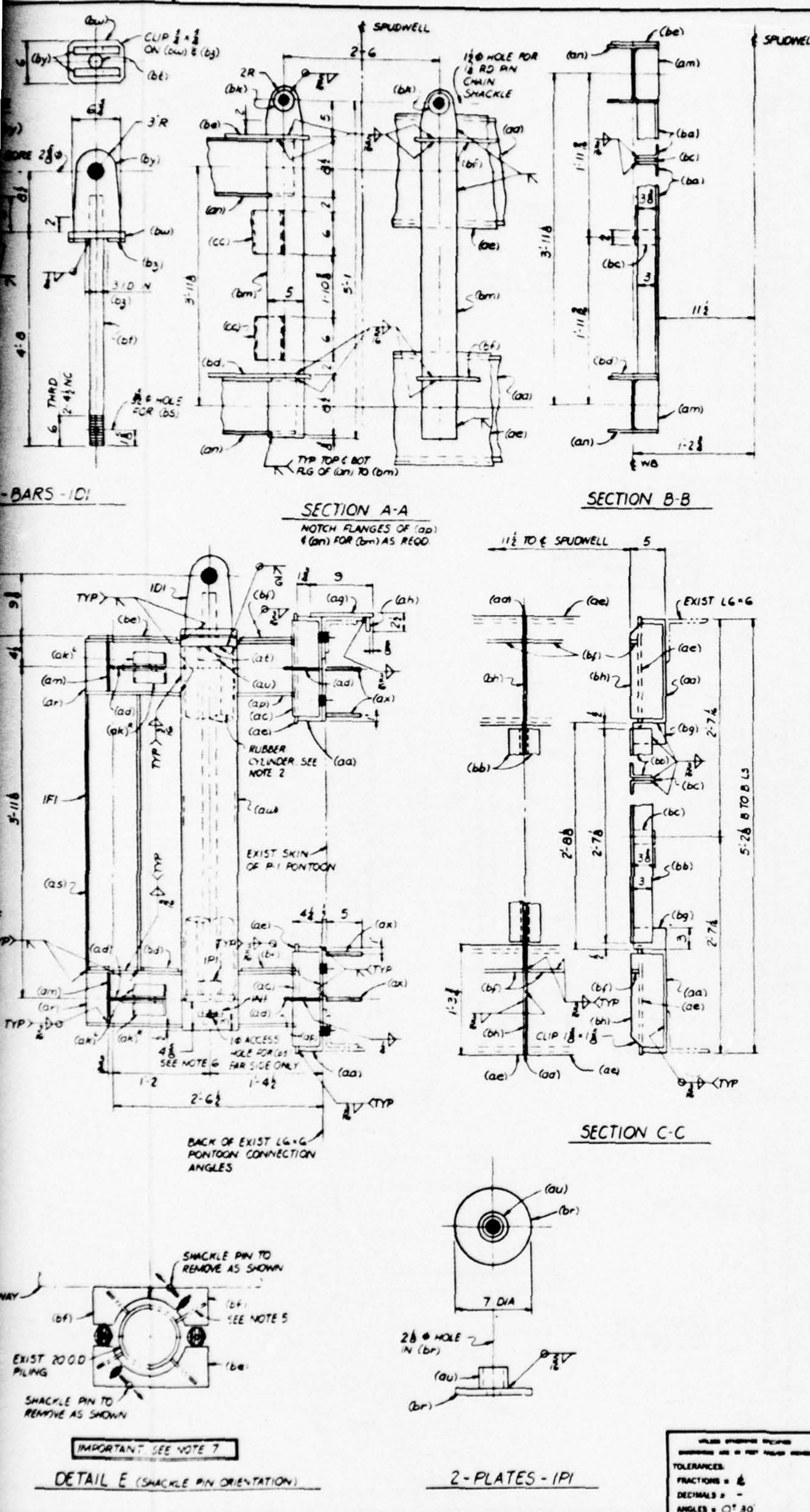


Figure B-2. External spudwell.

BEST A



REVISIONS				
SYM	DESCRIPTION	DATE	APPROVED	

NOTES

1. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM SPECIFICATION A36 OF MOST RECENT ISSUE UNLESS OTHERWISE NOTED.
2. THE AMPHIBIOUS & HARBOR DIV. CODE L55 WILL FURNISH THE RUBBER CYLINDER WHICH WILL HAVE THE FOLLOWING CHARACTERISTICS:
   - SIZE - 7" OD × 3" ID
   - LENGTH - 50"
   - DEFLECTION @ 50000 LBS - 5"
   - SPECIFICATION - ASTM-D2000 (LINE CALL-OUT) - 3BA720 B13C12D11F19
3. RUBBER SHALL CONFORM TO MIL-STD-417, TYPE S, CLASS SB, AND GRADE 705 FOR (bz).
4. CODE L55 WILL DETERMINE PLACEMENT OF EXTERNAL SPUDWELL ON EXISTING 3×15 CAUSEWAY. (FOR 3×15 CAUSEWAY SEE FIGURE 7-3 OF "PONTOON GEAR HANDBOOK NL EQUIPMENT P-SERIES" NAVFAC P-401.)
5. OPERATING PERSONNEL SHALL WELD WEDGE-SHAPED PLATES UNDER THE DIRECTION OF CODE L55 FOR MAKING THE CONNECTION OF SPUDWELL PLATES (bd) (be) & (bf) TO THE 20 OD PILING. SEE DETAIL A CEL DWG 74-35-1F.
6. COMPRESS THE RUBBER CYLINDER APPROXIMATELY 1/2" THEREBY PRELOADING IT TO ABOUT 5000 #.
7. AS PART OF THE ERECTION PROCEDURE 1 1/4 CHAIN SHACKLES MUST BE CONNECTED TO (bm) BEFORE THE WELDING OF THE WEDGE-SHAPED PS MENTIONED IN NOTE 5 IS ACCOMPLISHED. IT IS THEREFORE EXTREMELY IMPORTANT THAT THE RELATIONSHIP OF THE SHACKLE PINS & WEDGE-SHAPED PS BE AS SHOWN IN DETAIL E IN ORDER TO BE ABLE TO DISCONNECT THE SHACKLES AFTER THE PILE IS WELDED TO THE SPUDWELL.
8. SEE CEL DWG 74-35-1F FOR GENERAL PONTOON CAUSEWAY ASSEMBLY.

MARK	NO REQ'D	SIZE	LENGTH FT	LENGTH IN	REMARKS
bz	2	CLOSED CELL RUBBER 6 × 8 × 1/2 THICK, SEE NOTE 3			
by	4	BAR 6 3/4 × 1	0	11 1/2	
bw	2	BAR 6 × 1	0	8	
bt	2	BAR 2 Φ	5	1	AISI 1018, THRD ONE END
bs	2	EXTENDED PRONG COTTER PIN 3/8 × 4" LG, STEEL			
		2-BARS-1D1			
	2	HEXAGON SLOTTED NUT 2"-4 1/2 NC, SEMI-FINISHED FORGED, ASTM A325			
		2-NUTS-1N1 (NOT DETAILED)			
br	2	BAR 7 Φ	0	1	
au	2	TUBE 2 3/8 OD	0	2	2 1/8 ID, AISI-MT-1015 (COLD DRAWN)
		2-PLATES-1P1			
cc	2	BAR 6 × 1/2	0	6	
bm	2	BAR 5 × 1	5	3	
bk	4	BAR 3 Φ	0	0 5/8	
bh	2	BAR 4 5/8 × 5/8	1	2 1/4	CUT TO TIGHT FIT BEFORE WELDING
bg	2	BAR 3 × 5/8	0	4 5/8	
bf	4	PL 1/2 × 8 1/4	1	6 5/16	
be	ONE	PL 1/2 × 13	3	0 1/2	
bd	ONE	PL 1/2 × 13	3	0 1/2	
bc	2	BAR 2 × 5/8	0	3	
bb	2	L3 × 2 × 5/8	2	7 5/8	
ba	2	L3 × 2 × 5/8	3	2 5/8	
ax	8	BAR 5 × 1/2	0	8 1/2	
aw	2	TUBE 8 OD	4	7 5/8	7 1/4 ID AISI-MT-1020 (DOM)
av	2	TUBE 2 3/8 OD	0	2	2 1/8 ID AISI-MT-1015 (COLD DRAWN)
at	2	BAR 8 Φ	0	1	
as	2	W8 × 28	3	3 5/8	
ar	4	W8 × 28	1	3 11/16	
ap	4	W8 × 28	0	10 7/16	
an	2	W8 × 28	2	6 1/4	
am	6	BAR 3 5/8 × 5/8	0	7 5/8	
ak	8	L3 × 2 × 5/8	2	6 3/4	
ah	2	BAR 2 × 1/2	0	5 3/4	
ag	2	BAR 5 3/4 × 1/2	0	9	
ae	4	PL 5/8 × 15 3/4	2	6 1/4	
ad	8	BAR 5 × 5/8	0	6	
ac	4	BAR 3 5/8 × 5/8	1	2 1/4	CUT TO TIGHT FIT BEFORE WELDING
ab	8	BOLT 1 1/2 Φ	0	3 1/4	ASTM A325
aa	2	C15 × 33.9	6	1 1/2	
		ONE-FRAME 1F1			

BILL OF MATERIAL

PROJ NO YF 53 556 08.01.011	DEPARTMENT OF THE NAVY — NAVAL FACILITIES ENGINEERING COMMAND
CEL DWG NO 74-25-1F	CIVIL ENGINEERING LABORATORY, PORT HUENEME, CALIFORNIA
DES K MACK / DR K MACK / CHK / SUPV / DESIGN DIV DIR / DEPT HEAD / SATISFACTORY TO	NL EQUIPMENT EXTERNAL SPUDWELL FOR NL PONTOON CAUSEWAYS
APPROVED — OFFICER IN CHARGE — DATE	SIZE F — CODE IDENT NO 80091 — NAVFAC DRAWING NO
APPROVED — FOR COMMANDER NAVFAC — DATE	SCALE NONE — CONSTR CONTR NO — SPEC — SHEET 1 OF 1

UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN FEET AND INCHES
TOLERANCES:
FRACTIONS ± 1/16
DECIMALS ± -
ANGLES ± 0° 30'

J74-11, J75-19

REV 2

Figure B-2. External spudwell.

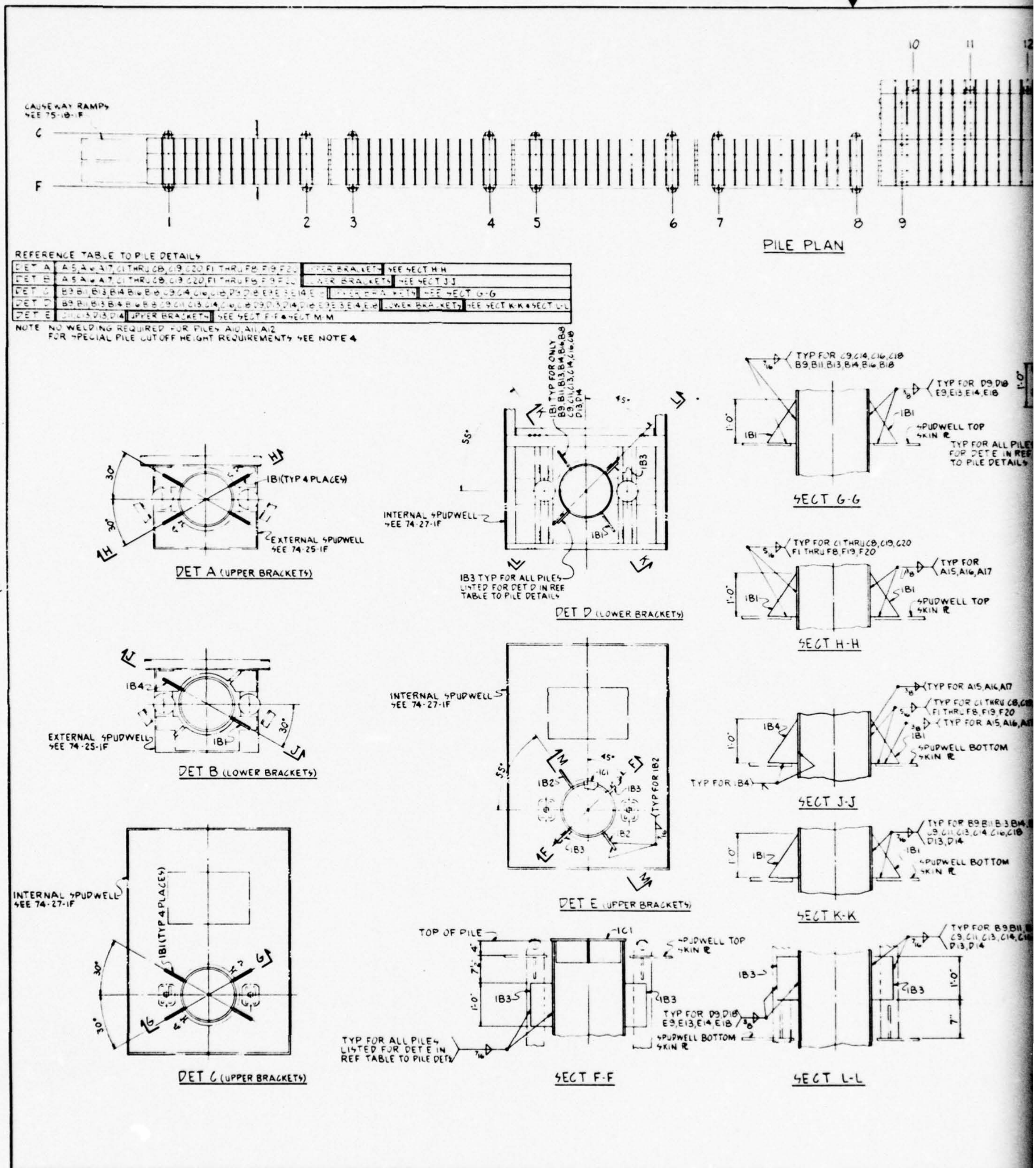
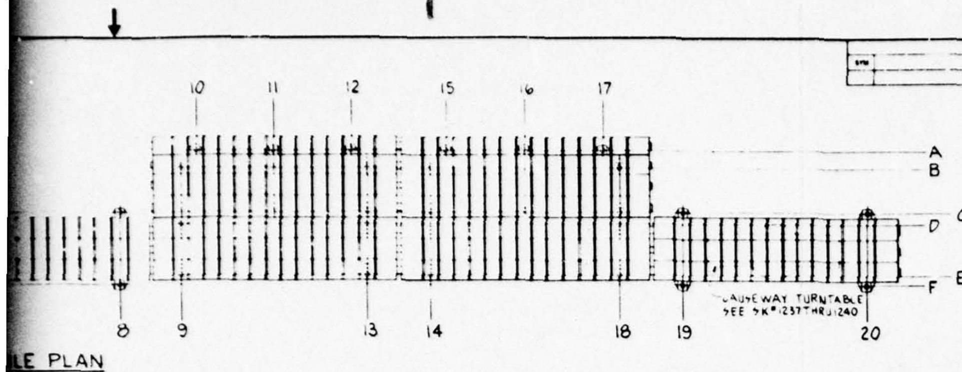
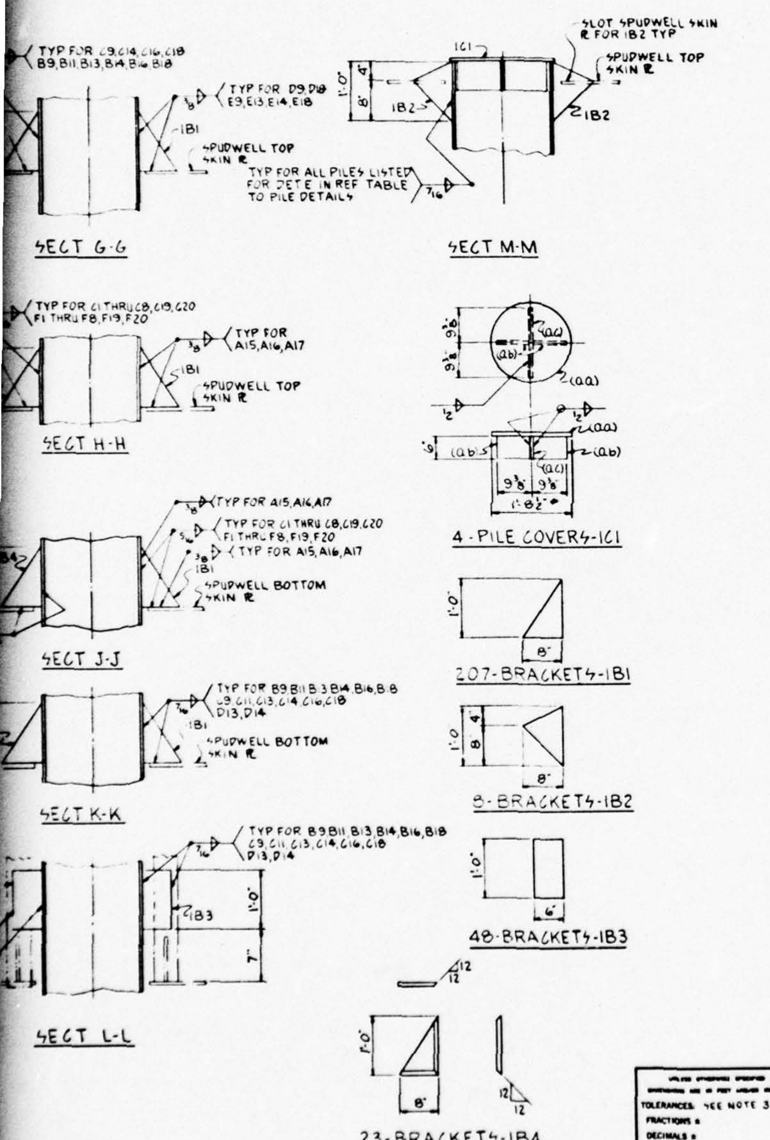


Figure B-3. Pile connectors.





NOTES

1. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM SPECIFICATION A36 OF MOST RECENT ISSUE UNLESS OTHERWISE NOTED.
2. USE AWS AS1 E70XX ELECTRODES FOR ALL WELDS.
3. BRACKETS 1B1, 1B2, 1B3 & 1B4 ARE DETAILED BASED ON PILES BEING ⊥ AND AT CENTER OF SPUDWELLS. BRACKETS MAY HAVE TO BE TRIMMED IN FIELD TO SUIT FIELD CONDITIONS.
4. THE FOLLOWING PILES ONLY HAVE LIMITATIONS ON THE PILE CUTOFF HEIGHT ABOVE THE TOP SKIN OF SPUDWELLS.
   B13, B14 & B16, CUT OFF HEIGHT 6'-0"
   C11, C13, D13 & D14, 0'-4" (SEE SECT F-F & SECT M-M)

MARK	NO REQ'D	SIZE	LENGTH FT	LENGTH IN	REMARKS
	23	BAR 8×½	1	0	BEVEL AS SHOWN
23-BRACKETS-1B4					
	48	BAR 6×½	1	0	
48-BRACKETS-1B3					
	8	BAR 8×⅝	1	0	
8-BRACKETS-1B2					
	207	BAR 8×½	1	0	
207-BRACKETS-1B1					
aa	4	BAR 6×½	1	6¾	
ab	8	BAR 6×½	0	9⅛	
aa	4	R 20½ ⌀	0	0½	
4-PILE COVERS-1C1					

BILL OF MATERIAL

UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN FEET AND INCHES
TOLERANCES: SEE NOTE 3
FRACTIONS ±
DECIMALS ±
ANGLES ±

PROJ NO
NCEL DWG NO 75-24-1F
DES B KAREN | DR M STERN
APPROVED DATE
APPROVED DATE
FOR COMMANDER NAVFAC

DEPARTMENT OF THE NAVY — NAVAL FACILITIES ENGINEERING COMMAND
NAVAL CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA

NL EQUIPMENT
SUPPORT BRACKETS & PILE COVERS FOR NL PONTOON CAUSEWAYS

SIZE F | CODE IDENT NO 80091 | NAVFAC DRAWING NO | CONSTR CONTR NO
SCALE NONE | SPEC | SHEET 1 OF 1

J75-34

Figure B-3. Pile connectors.

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Appendix C

IMPACT OF 40-FOOT CONTAINER ON
ELEVATED CAUSEWAY SYSTEM

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For the present and near time frame, the military will be concerned primarily with the 8 x 8 x 20-foot (2.44 x 2.44 x 6.10-m) container in the supply system. The 20-foot (6.10-m) container has a maximum gross weight of 44,800 pounds (20,300 kg). However, when the spreader bar, crane block, and other lifting hardware are included, the crane is actually required to make a lift of about 50,000 pounds (22,700 kg).

Future planning calls for the inclusion of the 8 x 8 x 40-foot (2.44 x 2.44 x 12.2-m) container into the military system. The 40-foot (12.2-m) container has a maximum gross weight of 67,200 pounds (30,500 kg), and, when the additional lift hardware is considered, a total crane lift approaching 75,000 pounds (34,000 kg) is required.

A study of the geometry of the system at the pierhead indicates a crane reach of at least 40 feet (12.2 m) is required; this was confirmed at the Coronado tests. Thus, the P&H 9125 crane* is the minimum required for the 40-foot (12.2-m) container.

LIMITING TRANSIT LOADS FOR CRANE

The elevated causeway provides a roadway from the beach to the pierhead for the container-handling crane. The loads seen by the causeway structure consist of local wheel loads on the pontoon deck and the total crane load applied to the basic structural frame.

Pontoon Deck

Wheel loads are most critical when applied directly to an individual pontoon. This problem is minimized since only one wheel or one dual wheel at a time can be applied to a pontoon. Also, the wheel spacing of most vehicles is about the same as the pontoon angles; therefore, the wheels can roll directly on the angles where the deck is much stronger.

The pontoon deck is designed to AASHO H20 wheel loads, that is, 4 tons (3,630 kg) to the front axle [4,000 pounds (1,800 kg) per single wheel] and 16 tons (14,500 kg) to the rear axle [16,000 pounds (7,260 kg) per single wheel]. The individual wheel load is the critical deck loading. An analysis of the pontoon structure agrees reasonably well with the H20 design criteria.

Since the wheel loads from large cranes frequently exceed 16,000 pounds (7,260 kg) per dual wheel, the pontoon deck requires structural reinforcement to accommodate these loads. For the elevated causeway tests, timber decking**was overlaid on the pontoons. Calculations indicate that the 4 x 12-inch (0.1 x 0.3-m) timber reinforcement increases the load capacity to about 21,000 pounds (9,530 kg) for a dual wheel and 19,000 pounds (8,620 kg) for a single wheel. To transfer cranes with higher wheel loadings, stronger reinforcement must be used, or the crane must drive on the causeway angles.

Consider the P&H 9125, whose wheel load distribution is shown in Figure C-1. With the boom laid forward, the rear axle loads reduce to 41,600 pounds (18,900 kg) per dual wheel for the crane under full operating load, and to 20,300 pounds (9,200 kg) for the crane with counterweights removed. Since the limiting value for the dual wheel is 21,000 pounds (9,530 kg) when the causeway is reinforced with 4 x 12-inch (0.1 x 0.3-m) timbers, the counterweights must be removed when the crane travels the causeway. With the boom trailing, as noted in the lower portion of Figure C-1, the 21,000-pound (9,530-kg) load limit will be exceeded with or without counterweights.

Structural Frame

The application of loads to the elevated causeway structural frame composed the bulk of the structural investigation. The determination of maximum crane loads that can be supported by the elevated causeway was of primary interest. The application of various

*The P&H 9125 crane is marginal. With a 70-foot (21-m) boom, its capacity is 74,000 pounds (33,600 kg) at 40 feet (12.2 m), or 75,000 pounds (34,000 kg) at 39.5 feet (12 m).

**Described in Volume V.

crane loads to a 3x15 pontoon causeway with 6-inch (15-cm) structural angles supported by four piles indicates a load capacity up to 130,000 pounds (59,000 kg). This load combination applies a stress level of 22,000 psi (151,000 kPa) to the structural angles when the crane is positioned in the most severe location. The failure analysis indicates a safety factor of about 2.6 for this load condition. By adding two piles at midspan, the capacity of the 3x15 causeway with 6-inch (15-cm) angles can be increased to 200,000 pounds (90,700 kg) with equivalent safety factors. There is little structural advantage to using more than six support piles.

Of significant importance to the COTS program is the fact that the P&H 9125 truck crane with a 140-ton (127,000 kg) rating can be reduced to a traveling weight of about 128,000 pounds (58,100 kg) by removing 62,000 pounds (28,100 kg) of counterweights. Therefore, a P&H 9125 without counterweights can be transported over a four-pile-supported 3x15 causeway with standard 6-inch (15-cm) angles.

The pontoon causeway system has an optional 8-inch (20-cm) structural angle assembly to increase its load-carrying capacity. The structural analysis indicates that a 3x15 causeway with 8-inch (20-cm) angles and supported on four piles can handle crane-type loads up to 235,000 pounds (106,600 kg) without exceeding 22,000 psi (151,000 kPa) stress. Adding two more pile supports at midspan increases the load capacity to 340,000 pounds (154,200 kg). These loads have safety factors to failure in the 2.5 range.

The P&H 6250 truck crane, which has an operating weight of about 353,000 pounds (160,100 kg), can transit the 3x15 causeway only by removing three counterweights that weigh a total of 90,000 pounds (40,800 kg). But even at a traveling weight of 263,000 pounds (119,300 kg), the crane must traverse a six-pile-supported causeway with 8-inch (20-cm) angles.

The structural frame data for various cranes are summarized in Figure C-2. Note that the traveling weights for minimum-sized cranes that can handle 40-foot (12.2-m) containers approach the limit of the 3x15 causeway with 6-inch (15-cm) angles on a four-pile support.

PIERHEAD

The pierhead sections where the crane operates require structural reinforcement. The extent of this reinforcement will depend largely on the type and size of crane used. Because of the number of cranes and numerous operating variables involved, it is impossible to generalize without an in-depth investigation.

The Army is planning* to proceed with the procurement of 34-ton (30,800-kg) tri-axle semi-trailers capable of hauling containers up to 40 feet (12.2 m) in length. The present turntable design will require only minor modifications (mainly, lengthening) to accommodate the longer trailer; weightwise the new trailers will be within the present lift capacity of the turntable.

*Container System Hardware Status Report, Project Manager, Army Container-Oriented Distribution System, July 1976.

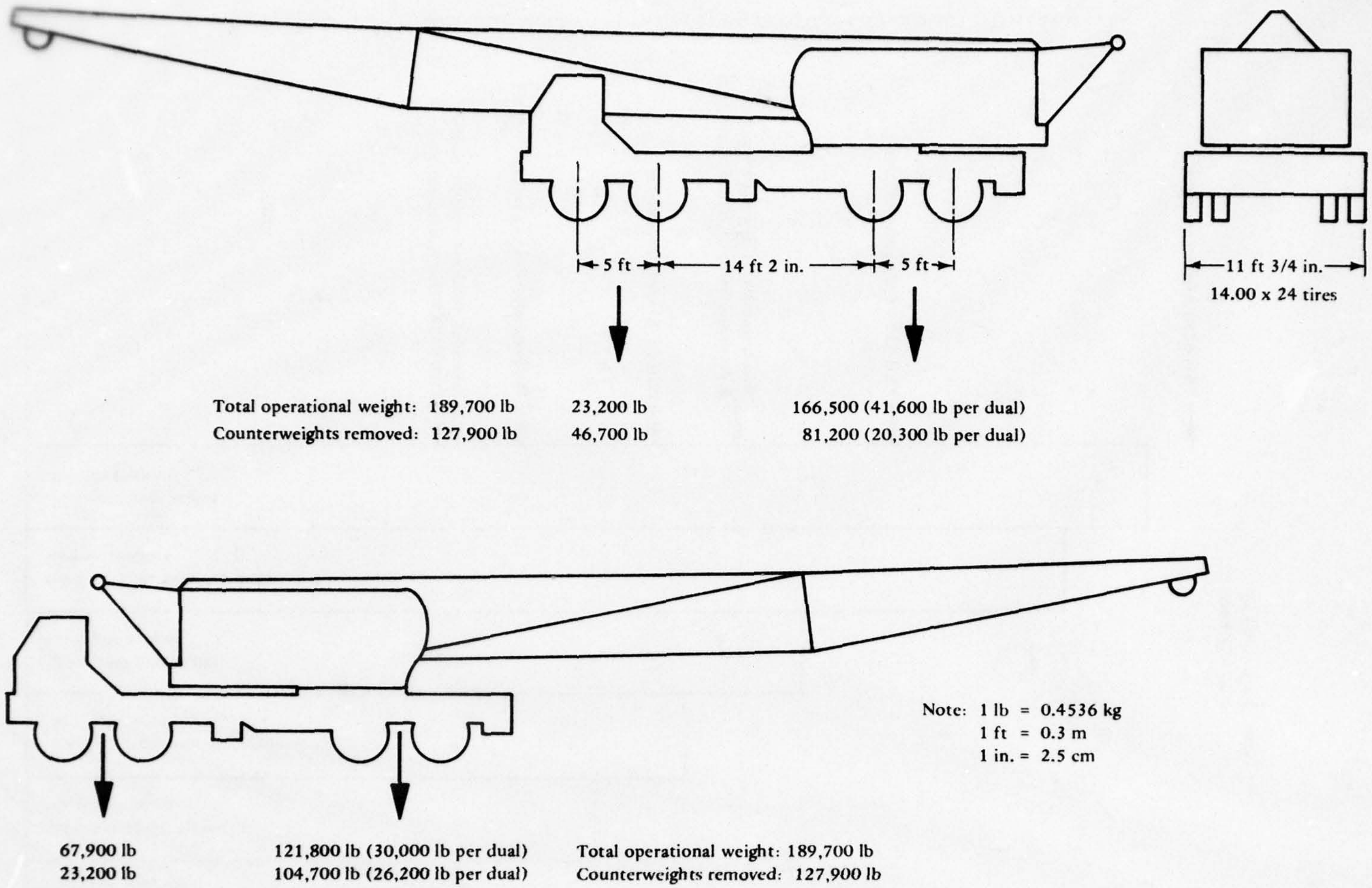

Figure C-1. Axle loads of the P&H 9125 truck crane.

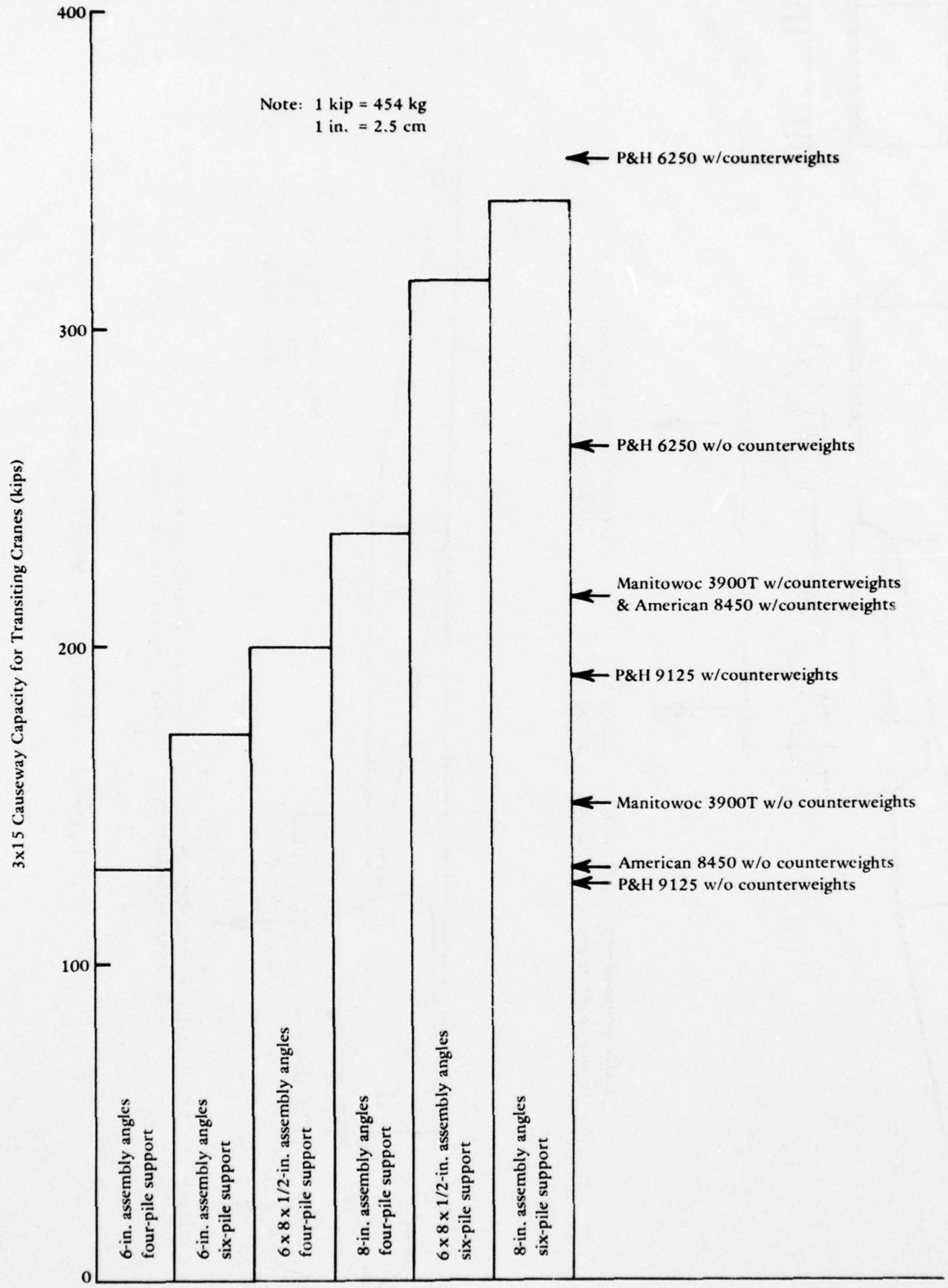

Figure C-2. Load capacity of different causeway configurations.

Appendix D

SYSTEM ALTERNATIVES AND COST ANALYSIS

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This appendix compares the elevated causeway system developed by CEL, designated "present system," with alternative elevated systems. The objective of this analysis was to compare the impact and cost of alternative elevated causeway systems to the total assault causeway system utilized and maintained by Amphibious Construction Battalion Units. Current policy dictates that each PHIBCB unit maintain enough assault causeways to operate two beaches. This breaks down to a 12-section floating causeway and a four-section causeway ferry for each beach – a total of 32 operational causeway sections.

Six alternative elevated causeway systems are presented in Table D-1; arguments for and against each system are given. The present system is broken down to its component level, and the functional role of these components is presented in Table D-2. Finally, alternatives to each component in the present system are presented in Table D-3. The probable impact of the various component alternatives on the present system is given in the table.

From the information given in Tables D-1 through D-3, CEL judges the "present system" to be the best technical and logistical solution. This elevated causeway system consists of seven approach sections* and four pierhead sections. An approach section differs from a standard assault causeway by the use of 8-inch (20.4-cm) causeway angles, timber decking, and attachment points for four external spudwells. The pierhead section extras include 8-inch (20.4-cm) angles, six internal spudwells, pontoon angle reinforcements for crane operations, three attachment points for external spudwells for fender system, timber decking, and side connectors. Approach sections without spudwells can function as assault sections. Pierhead sections could be used as assault causeways, but they are about 15 percent less buoyant and have six internal spudwells that could hinder operations. The "present system" advocates the use of approach sections as assault causeways, but pierhead sections should be maintained separate from the assault system.

The basic ground rule of the cost analysis for the alternative systems was to maintain a 32-section assault system and an 11-section elevated system (seven approach, four pierhead sections). Table D-4 presents the component costs and system costs of each alternative system. The possibility of casualties in an operation always exists; therefore, the cost of providing backup units is provided in Table D-5. The estimated cost data used to compute system costs are presented in Table D-6.

Of the four possible elevated systems, the integrated system is the least costly, while the separate assault/elevated system is the most costly. The cost of an all external spudwell system is about $170,000 less than an all internal system; however, an all external system has not been fully developed and would require additional development costs and time. Notably, the internal spudwell system and the external spudwell system assume that *all* assault causeways are converted to a dual assault/elevated role. Therefore, Table D-5 shows the option of converting only 14 sections to an elevated role.

*The number of approach sections could be more or less depending on the beach gradient and tidal range; e.g., a beach gradient of 1:50 with a 6-foot (1.83-m) tide requires 8 approach sections.

Table D-1. System Alternatives

Elevated Causeway System	Considerations
A. Develop a *separate* elevated causeway system using the basic NL pontoon system.	Technically ideal. Developmental costs for prototypes comparable to present approach. Would enlarge operating forces inventory with introduction of several single-purpose pontoon constructions. Good solution, but goes beyond present requirements.
B. Develop a *new pontoon system* to include elevated causeway mission.	Costly, but probably best solution technically and logistically in the long run. R&D costs are estimated at $14 million for a start date of FY-78. Escalation costs are included in the estimate. The new system could not produce an interim solution in the immediate future (next 5 to 7 years). Good solution, but goes beyond present requirements.
C. "Present Approach" – Develop a system that maximizes present NL pontoon system, but augmented by special developments.	Judged to be the best solution technically and logistically within the developmental constraints and criteria. Special purpose pontoon construction reduced to about 35% of that required in System A. Basically the additions are special NL pontoon sections for the pierhead.
D. Modify *all assault causeway sections* to include elevated causeway developmental features *with internal spudwells.*	Considered initially as a candidate system. Technically feasible; provisions would have to be incorporated to provide fendering interface between lighter and causeway. More costly to provide all sections with elevating capability and necessary reinforcements built-in; loss of buoyancy due to spudwells would be 8 to 13% (4 or 6 spudwells). Added cost plus buoyancy loss judged to be a severe penalty for a secondary capability for the causeways, vis-a-vis its primary assault mission.
E. Modify *all assault causeway sections* to include elevated causeway developmental features *with external spudwells.*	Technically unsound at pierhead where sections are required to be secured in side-by-side position for lateral stability. Substitute "make-do" solutions would be inferior, more complex, and difficult to execute, and finished construction would probably not meet design criteria. Considerable amount of field welding would be required. System not recommended.
F. *Modify* required number of *assault causeway sections at the AOA* to provide elevating capability.	Logistically unattractive and probably unacceptable because of time and resources limitations.

Table D-2. "Present Approach" System

Component	Function
Internal spudwells	Used for support piling. Replaces P1 pontoon in causeway on one-for-one basis. Used in pierhead where sections are side by side. Six spudwells are required for each pierhead section. The spudwells are installed during the initial construction of the pierhead section.
External spudwells	Used for pile support of the approach causeway from the beach to the pierhead. A minimum of four spudwells is required for each section. The spudwells may be installed on assault causeways when needed; assault causeway will require modification to accept the spudwells. Also used on the pierhead to support the fender system. Allows two-way traffic without cutting off piles extending above deck.
Side connectors	Used in the pierhead construction (positioning alignment and pierhead expansion) and for structural integrity to assure the pierhead acts as a unit in resisting lateral forces. The side connectors are incorporated in the pierhead sections.
End connectors	Used to connect sections end to end. Same function as in floating mode.
Fender system	Provides an interface between elevated causeway and lighterage. Consists of a single string of pontoons faced with commercial cushions to absorb lighterage impact loads.
Crane reinforcement at pierhead	Used to strengthen the pierhead for supporting the heavy vertical loads associated with the crane container-handling operation. Also used to more favorably distribute the loads.
Crane reinforcement for approach sections	Used to favorably distribute the crane load on its transition across the approach causeway to the pierhead. The reinforcement normally consists of timber decking.
Pile-to-causeway connection	Used for structural integrity to permanently connect the section to the support piles. Used to develop both vertical and lateral resistance to loads.
Turntable section	Section seaward of the main pierhead used to turn the trucks around.
Mooring components	Used for mooring lighters alongside and to the fender system and/or the elevated causeway.
Launching angles (modified)	Used to make the sections side-loadable on Class 1179 LST.
Beach transition ramp	Used to bridge the gap between the beach and the end of the causeway.
Construction equipment	Used for installation and erection of the elevated pier. Major components include a 35-ton construction crane, a 5-ton auxiliary hoist, piles and driver, and the lift (jacking) system.

Table D-3. System Component Alternatives

Component	Alternative Courses of Action	Impact on System
Internal spudwell	1. Assemble into pierhead sections.	Preferred; best technical solution. Extra sections to ship, over and above assault causeways.
	2. Assemble into a select number of assault causeways during initial construction.	To approach the preferred solution, should also consider adding crane reinforcement, side connectors, and fender system attachment. Buoyancy loss about 13%.
	3. Assemble into assault sections at AOA.	Time required to put system into operation too great. Assembly required ashore.
	4. Assemble into all assault causeways during initial construction. Use all internal spudwells.	Requires alternate solution for supporting fender system. More costly to provide all sections with elevating capability; also buoyancy loss due to spudwells.
	5. Eliminate; use all external spudwells.	See External Spudwell, Alternative 2.
External spudwell	1. Attach to assault sections as needed (for approach sections to pierhead).	Preferred. Requires assault causeways to be modified to accept the spudwell. Modify all or a select number earmarked for elevated causeway function.
	2. Use all external, including pierhead.	As above, alternative 1 modifications required to accept spudwells.
	(a) Pierhead one section wide	(a) Degrades system, less safe; less stable, more vulnerable to impact loads; necessary to relocate turntable, revise crane outrigger supports, and modify fender system. Does not meet development criteria.
	(b) Pierhead with external spudwells staggered to bring sections closer together in side-by-side position.	(b) Incompatible with side connectors; would make pier construction more difficult and hazardous; requires bridging sections for gap between sections; destroys structural integrity for lateral stability.
	3. Eliminate, use all internal.	See Internal Spudwell, Alternative 4.
Side connector	1. Assemble into pierhead sections.	Preferred, best technical solution.
	2. Assemble into a select number of assault causeways during initial construction.	See impact statement for Internal Spudwell, Alternative 2.

continued

Table D-3. Continued

Component	Alternative Courses of Action	Impact on System
Side connector (cont'd)	3. Install in assault sections at AOA.	Too time consuming. Requires installation on shore.
	4. Eliminate.	Not feasible; destroys structural integrity; makes pierhead construction more difficult and hazardous.
Fender system	1. Install external spudwell for fender system into pierhead sections at AOA.	Preferred. Pierhead sections must be configured to accept the external spudwells.
	2. Install external spudwells for fender system at time of initial assembly of pierhead sections.	Requires transport by means other than side-carry.
	3. Eliminate.	Requires new fendering concept that is apt to be more complex and difficult to install; e.g., mooring dolphin. Requires providing other vertical support for crane lost by eliminating the fender system external spudwells.
Crane reinforcement of pierhead	1. Install in pierhead sections.	Preferred; best technical solution.
	2. Install in select number of assault causeways (earmarked for pierhead) during initial construction.	See impact statement for Internal Spudwell, Alternative 2.
	3. Install in assault sections at AOA.	Too time consuming; requires installation on shore.
	4. Install in all assault sections.	Costly; unwarranted addition to all causeways.
	5. Design special crane cribbing to be installed after elevating.	Adds additional effort and time at AOA to make system operational.
	6. Eliminate.	Technically unacceptable.
Crane reinforcement for approach sections	1. Install timber decking on assault sections. (a) Prior to shipment to AOA. (b) At AOA.	Either Alternative (a) or (b) is workable; a logistics decision.
	2. Design and fabricate a stronger deck.	Costly approach to provide elevated causeway capability; stronger deck not required for most applications. Not a solution for existing pontoons.

continued

Table D-3. Continued

Component	Alternative Courses of Action	Impact on System
Pile-to-Causeway connection	1. Mechanical connection.	Preferred solution still in exploratory stage.
	2. Plate weldments executed in the field (present system).	Structurally satisfactory; requires more welding than operating force may be able to furnish.
Turntable section	1. Install turntable on assault causeway at AOA.	Preferred solution. Use external spudwells on turntable section.
	2. Install on separate section prior to shipping to AOA.	Requires a separate, special section; transported by other than side-carry.
	3. Eliminate turntable.	Degrades system to below acceptable, or requires significant expansion of pierhead.
Mooring components	1. Install bitts.	Either Alternative (a) or (b) is workable; a logistic decision.
	(a) At AOA.	
	(b) Prior to shipping.	
Launching angles	1. Install modified units to fit on sections.	Arguments pro and con are the same as for other devices installed on sections.
	2. Assault causeways are retrofitted to function as elevated sections at AOA.	Launching angles to be removed or modified to accommodate other hardware to convert section into elevating capability. All other pros and cons of using assault causeways apply.
	3. Eliminate.	Sections must be shipped by means other than side-carry.
Beach transition ramp	1. Present ramp.	Preferred workable solution. Installed at AOA.
	2. Use standard beach section.	Will require sand ramp to effect transition.
Construction equipment	1. Present setup.	Workable solution. No known workable alternative. Minor variations are possible, depending on logistics.

Table D-4. Component Costs of Alternative Elevated Causeway Systems

Component	Cost per Unit ($1,000)	Cost ($1,000) of Units for – Assault System		Integrated Assault/Elevated System		Assault/Elevated System (all internal spudwells)		Assault/Elevated System (all external spudwells)		Separate Assault and Elevated Systems	
		No. of Units	Cost	No. of Units	Cost	No. of Units	Cost	No. of Units	Cost	No. of Units	Cost
Causeway section											
Beach	75	2	150	2	150	2	150	2	150	2	150
Offshore	90	2	180	2	180	2	180	2	180	2	180
Intermediate	67	28	1,876	21	1,407	–	–	–	–	28	1,876
Intermediate with fender spudwells	115	–	–	–	–	2	230	–	–	–	–
Intermediate modified with internal spud-wells	102	–	–	–	–	26	2,652	–	–	–	–
Intermediate modified for external spud-wells	90	–	–	–	–	–	–	28	2,520	–	–
Approach/assault	96	–	–	7	672	–	–	–	–	7	672
Pierhead	109	–	–	4	436	–	–	–	–	4	436
Piles, fender, lift system, turntable	400	–	–	1	400	1	400	1	400	1	400
External spudwells[a]	3.3	–	–	–	–	–	–	58	191.4	–	–
Total System		–	2,206	–	3,245	–	3,612	–	3,441	–	3,714

[a] Applies only to the intermediate modified, all-external spudwell system. For all other systems the spudwells are costed with the sections.

Table D-5. System Composition Options and Costs

System	Section Additions[a]	Section Deletions[a]	Cost ($1,000)
Integrated[b]	2 Approach 1 Pierhead	2 Intermediate	3,412
All Internal Spudwells[c]	12 Intermediate	12 Intermediate modified with internal spudwells	3,192
All External Spudwells[c]	14 Intermediate	14 Intermediate modified for external spudwells	3,119
Separate Assault/Elevated[b]	2 Approach 1 Pierhead	–	4,015

[a]To the systems of Table D-1.

[b]Additional elevated capability provided for contingencies.

[c]Only 14 sections modified for elevated capability.

Table D-6. Unit Costs and Weights for Elevated Causeway Components

Component	Cost ($)	Weight (lb)
External spudwell (ea)	3,300	1,300
Internal spudwell (ea)	4,500	3,200
Side connector (per section)	1,100	900
Reinforcement for crane (per section)	1,000	200
Deck reinforcement, timber (per section)	6,000	14,000
Reinforcement for external spudwell (ea)	1,000	200
8-inch pontoon angle (per section)	6,000[a]	7,000[a]
1 x 15 string with 8-inch angle and fender	50,000	50,100

[a]Additional cost and weight over standard 6-inch pontoon angle in section.

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NAVFACENGCOM CDR L K Donovan, Alexandria VA; Code 043 Alexandria, VA; Code 044 Alexandria, VA; Code 0451 Alexandria, VA; Code 0453 (D. Potter) Alexandria, VA; Code 0453C, Alexandria, VA; Code 0454B Alexandria, Va; Code 04B3 Alexandria, VA; Code 04B5 Alexandria, VA; Code 06, Alexandria VA; Code 101 Alexandria, VA; Code 1023 (T. Stevens); Code 104 Alexandria, VA; Code 2014 (Mr. Taam), Pearl Harbor HI; PC-22 (E. Spencer) Alexandria, VA; PL-2 Ponce P.R. Alexandria, VA

NAVFACENGCOM - CHES DIV. Code 405 Wash. DC; Code FPO-1 (C. Bodey) Wash. DC; Code FPO-1 (Ottsen) Wash. DC; Code FPO-1SP (Dr. Lewis) Wash. DC; Code FPO-1SP13 (T F Sullivan) Wash. DC; Code FPO-IP12 (Mr. Scola). Washington DC
NAVFACENGCOM - LANT DIV. RDT&ELO 09P2. Norfolk VA
NAVFACENGCOM - NORTH DIV. Code 09P (LCDR A.J. Stewart); Code 1028. RDT&ELO. Philadelphia PA; ROICC. Contracts. Crane IN
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NAVFACENGCOM - WEST DIV. Code 04B; Code 2011; O9P/20
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NAVOCEANSYSCEN Code 3400 San Diego CA; Code 409 (D. G. Moore). San Diego CA; Code 52 (H. Talkington) San Diego CA; Code 6565 (Tech. Lib.). San Diego CA; Code 7511 (PWO) San Diego. CA; Code 8644B (R. Jones) San Diego CA
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